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BY

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INTRODUCTION.

THE author of this work has been led to its publication chiefly through the desire of many professional friends, and he would observe, in offering it to the profession, and others connected with the measurement of water, that it has been his aim not to produce a *treatise* on Hydraulics, for many have been written and leave little or nothing to be desired, but rather to introduce a number of *Hydraulic Formulæ* applicable to almost all cases which may be met with in the actual practice of the Hydraulic Engineer.

Neither does the author lay claim to pure originality; many of the formulæ have been simplified from other authors, and where they have been inserted full proof of their accuracy has been afforded to the author during an extensive practice.

Some of the computations are purely original, notably those for distribution pipes in towns.

The Appendix, No. 1, has been added chiefly for the purpose of enabling miners, and others engaged in sluicing operations or in the use of water for other purposes, to check the quantity of water supplied them, and it is hoped that the general arrangement of the whole work is such as to afford assistance, not only to the professional Hydraulic Engineer, but to those engaged in the use of the water supplied them.

Appendix II. contains numerous worked-out exercises which it is hoped will enhance the value of the work.

T. W. STONE.

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CHAPTER I.

PLATE I.

OF THE FLOW OF WATER OVER WEIRS.

THE velocity of a falling liquid without friction is given by the formula

$$V = \sqrt{2gH}, \qquad [1]$$

in which V equals the velocity in feet per second, g equals the force of gravity, and H equals the height in feet fallen; the value of g varies slightly in different latitudes, but for the purpose of hydraulic calculations it may be taken as equal to 32·2015, and consequently $2g$ in the following pages is assumed at 64·403.

The height due to a given velocity is expressed by the formula

$$\frac{V^2}{2g} \text{ or } \frac{V^2}{64 \cdot 403}, \qquad [2]$$

the notation remaining the same as for formula [1]. To find, therefore, the theoretical velocity of water flowing over a weir, the formula

$$V = \sqrt{2gH} \qquad [3]$$

may be modified to

$$8 \cdot 025 \times \sqrt{H}. \qquad [4]$$

For finding the discharge, however, it is necessary, owing to the contraction of the fluid vein and friction, to reduce the

sectional area of the water to $\frac{2}{3}$rds of that given by the measurement from the sill of the weir to *still* water, and so multiply by a coefficient determined by experiment according to the conditions under which the weir is placed, the area and width of the approaching channel, the thickness and form of the crest and weir basin. For it is evident if the width of the weir be the same as that of the channel, no contraction will occur at the ends, but if the weir is much narrower than the approaching channel, then contraction occurs, which, according to Francis's experiments, is found to reduce the effective length 0·2 of an inch for each inch depth of overfall. A modification of Francis's rule for weirs with thin plates is

$$G = 2 \cdot 4953 \times (l - 0 \cdot 1\, n\, d) \times d^{1 \cdot 5};$$

where

$G =$ gallons per minute;
$d =$ depth in inches;
$n =$ number of contractions (usually two).

For the actual velocity then the formula [2] will become

$$8 \cdot 025 \times \sqrt{H} \times c, \qquad [5]$$

and for the actual discharge

$$D = 8 \cdot 025 \times \sqrt{H} \times c \times \tfrac{2}{3} A; \qquad [6]$$

in which

$c =$ the coefficient according to circumstances;
$A =$ the area in square feet;
$D =$ the discharge in cube feet per second;
$H =$ the head in feet.

Tables II. to VI. inclusive show the generally received coefficients for weirs and orifices of different forms. In reference to the experiments of Mr. Blackwell, I have altered the coefficients given in some works by giving their full value. The sectional area of water as taken in these works is incorrect and confusing. It should be only $\frac{2}{3}$rds of that

area, and the full coefficient of experiment taken. From these tables it would appear that a coefficient of about 0·615 to 0·62 may be taken for notches in thin plates with square edges if the width above the notch is much wider than the width of the notch itself. If the edges are bevelled, as in the diagram, Plate I., the coefficient rises to 0·667, and the discharge can be calculated by the formula

$$\tfrac{2}{3}\sqrt{d} \times 5{\cdot}34 \times A = D\,; \qquad [7]$$

in which

d = depth over sill measured to still water, in feet;
A = area in square feet;
D = discharge in cube feet per second.

And as this is a formula in ordinary use where gauges fixed according to the diagram are placed across streams for the purpose of comparison with rainfall, I have calculated a Table, No. I., showing the quantities passing over a weir one foot wide for every $\frac{1}{16}$th of an inch up to 12 inches deep, and then for every $\frac{1}{4}$th up to 24 inches, which is the highest flow which can be obtained with accuracy from this formula [7], an alteration in the coefficient occurring at greater depths. This table only applies where there is no velocity of approach, and where the form of the weir and approaches are as in the diagram; in other cases formula [6] must be used and the proper coefficient applied.

Of Velocity of Approach.

Where the water approaches the crest of the weir with a sensible velocity, the discharge is that due to the *difference* of discharge between *two* weirs, the one having the discharge calculated from the measured head *plus* the head due to the velocity of approach, and the other calculated from the head due to the velocity of approach only.

Mr. Neville gives a rule and works out an example, page 104, second edition, in which the notch is 7 feet long,

head 8 inches, and velocity of approach $16\frac{1}{4}$ inches per second, and from this, by equation [42], he calculates the discharge as 13·573 cubic feet per second. This rule is, however, complicated, and by the rule I have given the discharge would be as follows, taking ·628 as the coefficient:—

$$\begin{array}{lr} & \text{Cube feet per second.} \\ 8{\cdot}025 \times \sqrt{0{\cdot}69781} \times 0{\cdot}628 \times \tfrac{2}{3} \times 4{\cdot}88 \quad = & 13{\cdot}672 \\ 8{\cdot}025 \times \sqrt{0{\cdot}03115} \times 0{\cdot}628 \times \tfrac{2}{3} \times 0{\cdot}21875 = & 0{\cdot}129 \end{array} \quad [8]$$

Difference which equals discharge in cube feet per second .. .. } 13·543

The head due to velocity of approach is found by the formula

$$\frac{\left(\frac{V}{c}\right)^2}{64{\cdot}403} = \text{head in feet} \qquad [8\text{A}]$$

Feet per second.

for an observed velocity of $16\frac{1}{4}$ inches per second = 1·35416
Therefore

$$\frac{\left(\frac{1{\cdot}35416}{{\cdot}956}\right)^2}{64{\cdot}403} = 0{\cdot}03115, \text{ the head in feet required;}$$

assuming a coefficient of 0·956 for the channel above the weir, which coefficient varies of course according to the description of channel above the weir.

Professor Downing gives a rule where L equals length in feet—

$$D = c \times 5{\cdot}35 \times L \times \sqrt{H^3 + 0{\cdot}03494 \times V^2 \times H^2}. \quad [9]$$

The discharge by this formula, taking $c = 0{\cdot}628$, would be equal to 13·1712 cube feet per second. I consider, however, that [8] is the best and simplest to adopt in practice.

Of the Flow of Water over Submerged Weirs.

For a submerged weir, or in other words when the *tail water* rises *above* the sill of the weir, the head to be taken becomes the difference of the level of the head and tail waters, then the square root of this head multiplied by the head from the tail water to the sill of the weir, plus $\frac{2}{3}$rds of the difference in level between the head and tail waters, equals the discharge in cube feet per second; or by formula,

$$d = 8\cdot025 \times \sqrt{H} \times (D + \tfrac{2}{3} \times H) \times L \times c; \quad [10]$$

in which

H = difference of level of head and tail waters in feet;
D = „ „ between tail water and sill of weir in feet;
c = coefficient according to circumstances;
L = length in feet;
d = discharge in cube feet per second.

When the water approaches the weir with a velocity, the discharge becomes

$$d = 8\cdot025 \times c \times L \times \{D \times \sqrt{H + H'} + \tfrac{2}{3} (H + H')^{1\cdot5} - H'^{1\cdot5}\} \quad [11]$$

in which H' equals the head in feet due to the velocity of approach taken in feet per second.

The index $1\cdot5$ or $\frac{3}{2}$ denotes the cube of the square root of the expressions to which it is attached, and is easily found by multiplying the logarithm of the number to which the index is attached by that index.

For example, what is the value of the expression

$$4^{\frac{3}{2}} \text{ or } 4^{1\cdot5}?$$

$$\begin{array}{rr} \text{Log of } 4 = & 0\cdot60206 \\ & 1\cdot5 \\ \hline & 301030 \\ & 60206 \\ \hline & 0\cdot903090 \end{array}$$

corresponding to number 8;

consequently

$$4^{\frac{3}{2}} \text{ or } 4^{1\cdot5} = 8.$$

Example of Calculation for Submerged Weir.—What is the discharge over a submerged weir (Plate I.) 40 feet long, and with the heads as in diagram, and an observed velocity of approach of 2 feet per second?

First, the head due to velocity of approach is, taking the coefficient of 0·956, to reduce the observed velocity to the theoretical velocity, equal to

$$\frac{\left(\frac{2}{\cdot956}\right)^2}{64\cdot403} = 0\cdot067 \text{ feet};$$

and taking the mean coefficient 0·628, we have for 1 foot of length

$$8\cdot025 \times \cdot628 \times 1 \times \{0\cdot8 \times \sqrt{1\cdot5 + 0\cdot067} + \tfrac{2}{3} (1\cdot5 + 0\cdot067)^{1\cdot5} - 0\cdot067^{1\cdot5}\}$$

for the discharge in cubic feet per second, equal to 11·68 cubic feet per second.

Then 11·68 × 40 = 467·2 cube feet per second for 40 feet wide. The coefficients of the submerged depth, D, and the upper part, H, generally differ according to experiments, and it is found that from 0·5 to 0·6 may be taken to represent the coefficient for the submerged depth, D, and 0·67 for the upper part, H. The formula for a submerged weir with these coefficients would become

$$L \times \sqrt{H} \times \{3\cdot56 \times H + 4 \times D\} \qquad [12]$$

= discharge in cube feet per second; with no velocity of approach, and all dimensions in feet. Where there is a velocity of approach and the coefficients differ, formula [11] should be used, using the proper coefficients to each portion of the expression.

Formula [10] is used on the supposition that $\frac{2}{3}$ H or $\frac{2}{3}$rds

of the difference of level in feet between the upper and lower waters, plus D, the depth in feet from the lower water to the sill of the weir together, multiplied by the length in feet, equals the area in square feet to be multiplied by the theoretical velocity, or

$$8 \cdot 025 \sqrt{\mathrm{H}},$$

for the theoretical discharge, which again multiplied by the coefficient of discharge, according to the nature of the case, gives the actual discharge in cube feet per second. And it therefore appears that when the water approaches the weir with a sensible velocity, the discharge over a submerged weir may be taken as the difference of discharge between two weirs, that for the first weir being calculated by formula [10], $\mathrm{H} + \mathrm{H}'$ being substituted for H, and for the second substituting for H in the formula [10] H', or the head due to the velocity of approach, and for the expression $(\mathrm{D} + \frac{2}{3}\mathrm{H}) \times \mathrm{L}$, an area equal to $\frac{2}{3}\mathrm{H}' \times \mathrm{L}$.

Taking the former example given to formula [11], we shall have for 12 inches wide, and taking the coefficient ·628 for both depths

$$\begin{array}{l} 8 \cdot 025 \times \sqrt{1 \cdot 567} \times (\cdot 8 + \tfrac{2}{3}\, 1 \cdot 567) \times 1 \times \cdot 628 = 11 \cdot 62 \\ 8 \cdot 025 \times \sqrt{0 \cdot 067} \times \tfrac{2}{3}(0 \cdot 067 \times 1) \times \cdot 628 = 0 \cdot 06 \\ \hline \qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad 11 \cdot 56 \end{array} \quad [11\mathrm{A}]$$

equal to discharge in cube feet per second for 12 inches wide. Therefore $11 \cdot 56 \times 40 = 462 \cdot 4$ cube feet per second for 40 feet wide, or a difference of only 1 per cent. from formula [11] with much less labour. A few more useful formulæ and examples are appended before closing this chapter.

Having given the depth of water over a weir and the length of the same, to find what depth of water will flow over a weir with a different length, the discharge remaining the same, the following formula may be used:—

$$d_1 = \left(\frac{w}{w_1}\right)^{\frac{2}{3}} \times d, \qquad [12\mathrm{A}]$$

in which w and w_1 equal the lengths of the two weirs, and d and d_1 the different depths passing over.

Example.—Water flowing down a river rises to a height of 0·87 feet on a weir 62 feet long; to what height will the same quantity of water rise on a weir similarly circumstanced 120 feet long?

$$\left(\frac{62}{120}\right)^{\frac{2}{3}} \times 0 \cdot 87.$$

$$\begin{array}{rr} \text{Log of } \cdot 516 = & -1 \cdot 71265 \\ \hline \div 3 = & -1 \cdot 90421 \\ \times 2 = & 2 \\ \hline & -1 \cdot 80842 \end{array}$$

Number corresponding to log ·644.

Then 0·644 × 0·87 = 0·567 or 6·8 inches = height required.

The fraction $\frac{2}{3}$ represents the square of the cube root of the expression to which it is attached. For example:—

What is the equivalent of $4^{\frac{2}{3}}$?

$$\begin{array}{rr} \text{Log of } 4 = & 0 \cdot 60206 \\ \hline \div 3 = & 0 \cdot 20068 \\ \times 2 & 2 \\ \hline & 0 \cdot 40136 \end{array}$$

Number corresponding to log 2·52.

$$\therefore 4^{\frac{2}{3}} = 2 \cdot 52.$$

It is also useful sometimes to find the horizontal distance to which a cascade from the crest of a weir will leap in the course of a given fall below the crest. It may be calculated by the following rule, supposing a coefficient of 0·667:—

$$v = 8 \cdot 025 \times \sqrt{H} \times \cdot 667, \qquad [12\text{B}]$$

which may be reduced to

$$v = 5 \cdot 35 \times \sqrt{H}, \qquad [13]$$

H being the height in feet to still water, and v the velocity in feet per second, remembering that to obtain the velocity the proper coefficient to suit the case must always be used.

Then to find the distance leaped by the cascade we have—

$$d = \frac{2v \times \sqrt{z}}{8 \cdot 025} = \frac{4}{3} \cdot \sqrt{z \times H}; \qquad [14]$$

in which

z = the given fall in feet;
d = the horizontal distance in feet;
H = the head to still water over the weir sill in feet;
v = the velocity in feet per second.

Example.—To what horizontal distance will the water leap in falling 10 feet from the sill of a weir over which the head to still water is 6 inches, using the coefficient of ·667?

$$\frac{4}{3} \sqrt{10 \times \cdot 5} = 2 \cdot 97 \text{ feet.}$$

To calculate the depth at which the sill of a weir should be placed *below* the original surface to ensure a given depth d_1 over the original surface, the weir being submerged, the following formula may be used:—

$$d_2 = \frac{D}{c \times l \times 8 \cdot 025 \times \sqrt{d_1}} - \tfrac{2}{3}\, d_1; \quad [17]$$

in which

c = the coefficient of discharge;
D = the discharge in cube feet per second;
l = the length of the weir in feet;

the other symbols as stated above.

Or by taking the velocity of approach, if any, into account,

$$d_2 = \frac{D}{c \times l \times 8 \cdot 025 \times \sqrt{d_1 + h}} - \tfrac{2}{3}\,\frac{(d_1 + h)^{1 \cdot 5} - h^{1 \cdot 5}}{\sqrt{d_1 + h}} \quad [18]$$

in which

h = the head due to velocity of approach.

Example.—For a river discharging 812·5 cubic feet per second, and 70 feet wide at surface, to what depth d_2 shall the sill of a weir be placed below the original surface to ensure the depth of water immediately above it being increased by 18 inches or 1·5 feet? We have then, taking ·628 as the coefficient of discharge, and as the weir is 70 feet wide, 11·6 feet per second will pass over each foot,

$$d_2 = \frac{11 \cdot 6}{\cdot 628 \times 1 \times 8 \cdot 025 \times \sqrt{1 \cdot 5}} - \tfrac{2}{3} \times 1 \cdot 5$$
$$= 1 \cdot 88 - 1 = 0 \cdot 88 = \text{height in feet required;}$$

the submerged weir must therefore be built 0·88 feet *below* the surface to raise the head 1·5 feet above the former level.

Taking the velocity of approach into account, the result becomes by formula [18], taking the value of h calculated from formula [2], and taking the theoretical velocity as equal to 2 feet per second $\frac{2^2}{64 \cdot 403} = 0 \cdot 062$ feet head. Then for 1 foot in length we have

$$d_2 = \frac{11 \cdot 6}{\cdot 628 \times 1 \times 8 \cdot 025 \times \sqrt{1 \cdot 5 + 0 \cdot 062}} - \tfrac{2}{3}\,\frac{(1 \cdot 5 + 0 \cdot 062)^{1 \cdot 5} - 0 \cdot 062^{1 \cdot 5}}{\sqrt{1 \cdot 5 + 0 \cdot 062}}$$

$$\therefore\ d^2 = 1 \cdot 84 - \tfrac{2}{3}\,\frac{(1 \cdot 5 + 0 \cdot 062)^{1 \cdot 5} - 0 \cdot 062^{1 \cdot 5}}{\sqrt{1 \cdot 5 + 0 \cdot 062}}$$

The latter part of this expression is best done by logarithms.

Log of 1·562 =	0·19368	Log of 0·062 =	−2·79239
× 1·5	1·5		1·5
	96840		796195
	19368		279239
	0·290520		−2·188585
Natural number	1·95	Natural number	0·015

Log of 1·562 =	0·19368
÷ 2 for sq. root	0·09684
Natural number	1·25.

Then the expression becomes

$$1\cdot84 - \tfrac{2}{3}\,\frac{1\cdot95 - 0\cdot015}{1\cdot25},$$

or

$$1\cdot84 - 1\cdot03,$$

or

$$d_2 \text{ is equal to } 0\cdot81 \text{ feet.}$$

The following formulæ give the discharge in cubic feet per second through notches of the various forms shown in the diagrams, Plate I.

For

A, $D = \frac{2}{15} \times c \times 8\cdot025 \times d^{1\cdot5} \times (2 \times t + 3\,b)$ [20]

B, $D = \frac{2}{3} \times c \times l \times d^{1\cdot5} \times 8\cdot025$ [21]

C, $D = \frac{2}{15} \times c \times 8\cdot025 \times d^{1\cdot5} \times \{(2 \times t) + (3 \times b)\}$ [22]

D, $D = c \times 8\cdot025 \times \frac{4}{15} \times t \times d^{1\cdot5}$; [23]

which for a right-angled triangle becomes

$$D = c \times 8\cdot025 \times \tfrac{8}{15} \times d^{2\cdot5}; \qquad [24]$$

for

E, $D = c \times 8\cdot025 \times 0\cdot9752 \times l \times d^{1\cdot5}$; [25]

and when the parallelogram becomes a square,

$$D = c \times 8\cdot025 \times \sqrt{l} \times 0\cdot34478 \times l^2. \qquad [26]$$

For

$$F, D = c \times 8\cdot025 \times \tfrac{8}{15} \times b \times d^{1\cdot5} \qquad [27]$$
$$G, D = c \times 8\cdot025 \times \sqrt{r} \times 0\cdot6103 \times A \qquad [28]$$
$$H, D = c \times 8\cdot025 \times \sqrt{r} \times 0\cdot9604 \times A \qquad [29]$$
$$I, D = c \times 8\cdot025 \times \sqrt{r} \times 0\cdot7324 \times A \qquad [30]$$

where

D is equal to discharge in cube feet per second;
c a coefficient according to circumstances (see Diagram);
d the depth in feet.
t width at top or surface of water in feet;
b „ bottom in feet;
r the radius in feet;
l the length in feet.

One example will suffice.

What is the discharge from a triangular notch, the width of water or *t* being equal to 2 feet and the depth or *d* equal to 1·73 feet, and taking a coefficient of 0·617 by [23] we have

$$\cdot617 \times 8\cdot025 \times \tfrac{4}{15} \times 2 \times 1\cdot73^{1\cdot5} = 5\cdot993.$$

Log of 0·617	= $\bar{1}$·79029	Log of 1·73 =	0·23805
„ 8·025	= 0·90444		1·5
„ ·266 or $\frac{4}{15}$	= $\bar{1}$·42488		119025
„ 2	= 0·30103		23805
„ $1\cdot73^{1\cdot5}$	= 0·35707		0·357075
	0·77771		
No. of log	5·993		

Therefore the discharge is equal to 5·993 cubic feet per second. It may be well to mention that the formula given by Professor Thompson for a right-angled triangle for discharges of from 2 to 10 feet per minute is as follows:—

$$Q = 0\cdot317 \times H^{2\cdot5},$$

in which

Q is equal to the quantity in cube feet per minute,
H ,, ,, head in inches,

and it appears that a coefficient of 0·617 used in the formula [24] of this work will give the same result as that obtained by the Professor in his formula. The advantage claimed for a triangular notch is that the coefficient probably remains the same for different depths and widths.

The diagrams in illustration of this chapter are given in Plate I., with the coefficients and formula I propose for each form.

CHAPTER II.

PLATE II.

ON THE DISCHARGE OF WATER THROUGH ORIFICES.

ALTHOUGH the velocity due to the height of water above the centre of gravity of an orifice is not strictly the mean velocity, still for all practical purposes it may be so taken, and the coefficients of discharge given in the tables comprehend the correction for the error arising from using the head from still water to the centre of gravity of the orifice, and this correction becomes inappreciable when the head exceeds three times the height of the orifice.

This *Theoretical* velocity is found to be equal to the expression

$$V = 8 \cdot 025 \times \sqrt{H},$$

and the *Theoretical* discharge equal to

$$D = 8 \cdot 025 \times \sqrt{H} \times A;$$

in which

V = theoretical velocity;

$H = \begin{cases} \text{head of water over the centre of gravity of the} \\ \quad \text{orifice measured to still water in feet;} \end{cases}$

D = theoretical discharge in cube feet per second;

A = area of the orifice in square feet.

Before entering into coefficients which should be used for obtaining the *actual* discharge in different cases, it may be well to give the general rules for finding the centre of gravity of the different shaped orifices which may possibly be met with in practice.

These are the square, the right-angled parallelogram, the

trapezoid, the triangle, the circle, the semicircle, and the polygon.

To find the Centre of Gravity of a Trapezoid (see Plate II.). Let

$$a \;= \mathrm{A\,L} = \mathrm{L\,D}$$
$$b \;= \mathrm{B\,K} = \mathrm{K\,C}$$
$$3\,c = \mathrm{K\,L};$$

then

$$\mathrm{K\,G} = c \times \frac{b + 2\,a}{b + a}.$$

Supposing G in the figure, Plate II., to represent the centre of gravity of which it is desired to find the position. For a triangle the centre of gravity lies upon the straight line which joins the summit to the middle of the base at one-third ($\frac{1}{3}$) of this line distance from the base; this line is easily calculated when the sides of the triangle are known, for in the diagram, Plate II., calling the side A B, a; the side A C, b; half the side B C or B D, c; and A D, d; we have $a^2 + b^2 = 2\,c^2 + 2d^2$.

Example.—Suppose we have a triangle each side of which is equal to 12″ or 1 foot, we have then $1^2 + 1^2 = 2$, and half the base is equal to $0 \cdot 5$, then $0 \cdot 5^2 \times 2 = 0 \cdot 5$;

$$\therefore 2 - 0 \cdot 5 = 1 \cdot 5$$

equals twice the square of the line A D on the diagram;

$$\therefore \sqrt{0 \cdot 75} = 0 \cdot 86.$$

Therefore $0 \cdot 86$ is equal to the length of the line A D, and the centre of gravity is distanced $\frac{0 \cdot 86}{3}$ or $0 \cdot 287$ feet from the centre of the base.

The centre of a circle is the centre of gravity.

The position of the centre of gravity in a semicircle is equal to

$$0 \cdot 4244 \times \mathrm{R}, \qquad [34]$$

where R is equal to the radius.

The centre of gravity in a regular polygon is at its geometrical centre.

It is also necessary in using the formula given to be able to find the areas of the figures enumerated easily, and though these of course may be obtained from any work on mensuration, it will render this work more complete for reference if the modes of finding the areas of the forms of orifice just given are inserted.

For the square. Square its side.

For the right-angled parallelogram. Multiply one side, the larger, by the less.

The trapezoid. Multiply the sum of the parallel sides by the perpendicular distance between them, and half the product is the area.

The triangle. Multiply the base by the perpendicular, and half the product is the area.

The circle. Square the diameter and multiply by 0·7854, and the product will be the area.

The semicircle equals half the area obtained for the full circle.

The segment of a circle less or greater than a semicircle.

Find the area of the sector that has the same arc as the segment; find also the area of the triangle whose vertex is the centre and whose base is the chord of the segment, then the area of the segment is the difference, or sum of these two areas, according as the segment is greater or less than a semicircle.*

The regular polygon. First find the interior angle by the following rule. From double the number of sides of the polygon subtract 4, multiply the remainder by 90, divide the product by the number of sides, and the quotient is the number of degrees in the interior angle; next find the apothegm thus. Multiply half the side of the polygon by

* To find the area of a sector of a circle, multiply the length of the arc of the sector by the radius, and half the product will be the area.

the tangent of half its interior angle, and the product is the apothegm.

Then for the area the following rule may be used. Find the continued product of the side, the number of sides, and the apothegm, and half the product is the area.

To find the area of an irregular polygon. Divide the polygon by means of diagonals into triangles and trapezoids, and find the area of the different figures and add them together for the total area.

When water issues from a circular orifice in a thin plate a contraction occurs at about the distance of half the diameter of the orifice; the diameter of this contraction is equal to 0·784, that of the orifice being 1, and the curve joining these two diameters may be taken as having a radius of about 1·22 times the diameter of the orifice, the velocity at the orifice itself is therefore less than the velocity at the contracted portion in the ratio of 1^2 to $0{\cdot}784^2$, or 1 to 0·615, and the actual velocity if the area be taken *at* the opening will become

$$8{\cdot}025 \times \sqrt{H} \times 0{\cdot}615. \qquad [35]$$

It has been proved from experiment, however, that this coefficient of 0·615 varies with the form of the orifice, the amount of head over it, its position in the sides of the vessel, and the proportion its breadth or diameter bears to the head of water upon it (see Tables II., &c.). The formula then becomes

$$D = 8{\cdot}025 \times \sqrt{H} \times C \times A; \qquad [36]$$

in which

D = discharge in cube feet per second;
H = head in feet from still water to centre of gravity of orifice;
C = a coefficient according to circumstances (see Plate II.);
A = area of orifice in square feet.

Submerged Orifices.

When the water from the orifice discharges into air, the head may be assumed as that from the centre of gravity of the orifice to the level of still water above the same, as before stated; but nothing could be more erroneous than to take this head when the orifice is either *wholly* or *partially* submerged. The head or depth in the first case becomes equal to the difference of the pressures, or the difference in height between the surfaces of the water on each side of the orifice. The discharge for a wholly submerged orifice is therefore

$$8 \cdot 025 \times \sqrt{H'} \times c \times A; \qquad [37]$$

in which H′ equals the difference of level from still water between the upper and lower waters (see Diagram 2, Plate II.).

When the orifice is partially submerged, the submerged depth is equal to the *difference* in level between the height from the bottom of the orifice to the surface of the upper water, *and* the height from the surface of the upper water to the surface of the lower water, and the remaining depth to the *difference* between the height of the upper water above the lower, *and* the height from the top of the orifice to the upper water, or in the Diagram 3, Plate II.

The submerged depth equal to $b - h$
And the remaining depth equal to $h - t$.

Calling, then, the submerged depth d and the remaining portion d_1, and h the difference of level of the still water on each side of the orifice, we have for the discharge of the submerged portion in cubic feet per second, with all dimensions in feet, equal to

$$c \times l \times d \sqrt{h} \times 8 \cdot 025, \qquad [38]$$

and for the remaining portion d_1

$$\tfrac{2}{3} \times c \times 8 \cdot 025 \times (h^{1 \cdot 5} - t^{1 \cdot 5}) \times l; \qquad [38\text{A}]$$

in which

c = the coefficient;
l = the length in feet;

and the other letters signify as stated above.

The discharges found by formulæ [38] and [38A] must then be added to obtain the total discharge in cubic feet per second.

This form of the equations is given as the coefficient c may be of a different value for the upper and lower depths. If, however, it is assumed the same, the following equation may be substituted for discharge in cubic feet per second, all dimensions in feet, and the letters having the same value as above.

$$D = c \times l \times 8{\cdot}025 \times \{d \times \sqrt{h} + \tfrac{2}{3} \times (h^{1{\cdot}5} - t^{1{\cdot}5})\}. \quad [39]$$

On Velocity of Approach.

When the water approaches a *fully* submerged or *unsubmerged* orifice with a sensible velocity, the following formula must be used :—

$$D = A \times 8{\cdot}025 \times \sqrt{H} \times c \times \sqrt{\left(1 + \frac{H_1}{H}\right)}, \quad [39A]$$

where

D = discharge in cubic feet per second;
A = area in square feet of the orifice;
H = head measured from centre of orifice to still water if not submerged, or difference of level between head and tail waters if submerged, in feet;
c = coefficient according to circumstances (see Plate II.);
H_1 = the head in feet due to the velocity of approach (see [8A]).

When a velocity of approach occurs in *partially* submerged orifices, substitute in the formula [39]

$$h + H_1 \text{ for } h,$$

and

$$t + H_1 \text{ for } t.$$

It is frequently required in connection with lock chambers on canals and rivers to find the time that the water takes to rise a certain height in the lock chamber when supplied from the canal, and also to ascertain the time in which the water will fall to a certain level in the chamber when it is required to empty the lock.

In the Diagram 4, Plate II., let it be required to know in what time the surface water K B will sink a given depth to L M; it is given by the formula

$$T = \frac{\frac{A}{4\cdot012 \times a} \times (\sqrt{H+F} - \sqrt{H})}{c}; \qquad [40]$$

in which

c = the coefficient;
T = the time in seconds;
A = the *horizontal* area of the vessel in square feet;
a = the area of the orifice O P in square feet;
F and H heights as shown in the diagram feet.

Example.—For a circular tank of 3 feet diameter, in what time will the water fall from K B to L M, or a height F equal to 5 feet, the total depth H + F being equal to 7 feet? The orifice O P being 1 foot diameter and taking ·628 as the coefficient, by formula

$$\frac{\frac{3^2 \times \cdot7854}{4\cdot012 \times 1^2 \times \cdot7854} \times (\sqrt{5+2} - \sqrt{2})}{0\cdot628} = 6\cdot42 \text{ Time in secs.} \qquad [40]$$

For the time the water takes to rise in a lock chamber to the level K L, when filled through an orifice O P from a canal or large chamber whose surface always remains at the same level, the time in seconds of rising from L M to K L is given by the formula

$$\frac{2 \times A \times \sqrt{F}}{c \times a \times 8\cdot025}, \qquad [41]$$

and therefore the time of rising from the centre of the orifice to L M equal to—

$$\frac{A}{4 \cdot 015 \times c \times a} \times (\sqrt{H} - \sqrt{F}). \qquad [42]$$

Supposing, therefore, the lower vessel to be at first empty, first find the contents in cube feet contained by the vessel below the centre of the orifice O P, and call it C, then the time of filling in seconds to the level of the centre of O P is given by the formula

$$\frac{C}{8 \cdot 025 \times c \times a \times \sqrt{H}}; \qquad [43]$$

where

H = the head in feet, as shown in the diagram;
a = area of the orifice in square feet;
c = a coefficient (see Plate II.).

The time, therefore, of filling up to any level L M is equal to the addition of the time found by [42] and [43] or

$$\frac{C}{8 \cdot 025 \times c \times a \times \sqrt{H}} + \left\{\frac{A}{4 \cdot 015 \times c \times a} \times (\sqrt{H} - \sqrt{F})\right\}. \qquad [44]$$

These formulæ apply to single locks, but the expressions for double locks are so easily deduced that it is unnecessary to repeat them here. Half the water only is used by providing double locks, but the expense is great, and it depends much upon the available supply of water whether single or double locks should be used; only half the water is used also by locking boats up and down alternately.

To find the quantity of water expended in passing a boat through a lock, subtract the amount of displacement from the cubic contents of the lock.

Orifice under Variable Head.

When the head on an orifice is variable, to find the constant head which would have given the same discharge in the same time the following formula may be used:—

$$H' = \left(\frac{H - h}{2(\sqrt{H} - \sqrt{h})}\right)^2;$$

in which

H' = the constant head in feet;
H = the head of the reservoir at the commencement of the flowing; and
h = the head in the reservoir at the end of the flowing.

Time taken to Empty a Reservoir under a Variable Head.

Take the areas in square feet of the contour line at each foot depth, and convert this into cube feet by multiplying by the depth of one foot, and find the time necessary to discharge this quantity by the formula and coefficient applicable to the discharging orifice; proceed in the same manner to the last depth, adding the several times together, not forgetting to multiply the last time *obtained* by 2 before adding it. The result of these additions will be the time taken to empty the reservoir.

The closer the contours are taken together, the more accurate will be the result.

CHAPTER III.

PLATE III.

ON THE DISCHARGE THROUGH SHORT TUBES.

WHEN an orifice is thickened or becomes a short tube of a length of about 2 diameters, the discharge is found to increase from that given by an orifice in a thin plate, the contraction in this case being about $0 \cdot 9$ when the diameter of the tube equals 1 and its proportional area $0 \cdot 9^2$ or $\cdot 81$, but this coefficient also varies as for an orifice in a thin plate (see remarks on causes of variation in coefficients, Chapter II.), and the formula becomes

$$8 \cdot 025 \times \sqrt{H} \times A \times c = D; \qquad [46]$$

in which

H = the head to still water from the centre of gravity of the tube;

A = the area of the tube in square feet;

c = a coefficient according to circumstances (see Plate III.);

D = discharge in cubic feet per second.

It will be seen that the *theoretical* formula for the discharge through a tube is exactly similar to that for a thin plate, but that the coefficient of discharge rises from about $0 \cdot 615$ in a thin plate to about $0 \cdot 817$ in a short tube of 2 diameters.

As the mean coefficient for short tubes with square edges, about 2 to $2\frac{1}{2}$ diameters in length, appears by the tables to be $0 \cdot 817$, I think in practice the formula [46] may be stated, to save labour, as below:—

$$G = \sqrt{H} \times d^2 \times 13; \qquad [47]$$

from which we obtain

$$H = \left(\frac{G}{d^2 \times 13}\right)^2, \qquad [48]$$

and

$$d = \sqrt{\frac{G}{\sqrt{H} \times 13}}; \qquad [49]$$

where

G = gallons per minute;
H = head in feet measured to the *centre* of the lower end of the tube; and
d = diameter in inches.

Submerged Tubes.

When the tube is wholly submerged, then H in the formulæ [46], [47], [48], [49], becomes the difference of level between the upper and lower waters. When the tube is partially submerged, the formulæ [38], [38A], [39] for thin plates may be used, not forgetting to increase the coefficient to 0·817 if the tube have square edges, and to 0·976 if the edges are rounded.

Velocity of Approach.

Where the water approaches the tube with a sensible velocity, the formulæ [39], [39A] may be used, substituting in formula [39]

$$h + H_1 \text{ for } h$$

and

$$t + H_1 \text{ for } t$$

(in which the letters have the same signification as in the article on thin plates), for partially submerged tubes, and increasing the coefficient to ·817 for square edges and 0·976 for rounded ones, or by formula.

For *partially* submerged tubes—

$$D = c \times l \times 8{\cdot}025 \times \{d \times \sqrt{h + H_1} + \tfrac{2}{3} \times (h + H_1)^{1{\cdot}5} - (t + H_1)^{1{\cdot}5}\},^* \qquad (49A)$$

* When the coefficient varies for the upper and lower depths in a *partially* submerged tube, use formulæ [38], [38A], substituting $h + H_1$ for h, and $t + H_1$ for t.

and for *wholly* submerged or *unsubmerged* tubes—

$$D = A \times 8\cdot025 \times \sqrt{H} \times c \times \sqrt{\left(1 + \frac{H_1}{H}\right)}, \quad (49\text{B})$$

the letters having the same signification as in formula [39A] Chapter II., and the coefficient c being taken as 0·817 for tubes with square edges, and ·976 with rounded edges. The coefficients generally admitted for tubes may be given as follows:—

When the ends next the reservoir are square edged, the coefficient is 0·817, as above.

When the ends next the reservoir are rounded, the coefficient increases up to 0·976.

When the pipe projects into the vessel, and has square edges, the coefficient is reduced to 0·705.

In tubes converging, the coefficient varies from 0·858 at 1° to 0·844 at 50°, the sectional area of the smaller end being taken in the formula [46].

For a divergent tube the coefficient increases from 0·858, for an angle of 1°, to about 0·885, and at 50° decreases from ·985 to ·872, the smaller area of the tube being used in both cases.

The discharge may even be increased over the theoretical discharge by a tube of the following dimensions (see Plate III.).

The coefficient in this case appears from Venturi's experiments to rise to 1·57, the area being taken at the narrowest portion.

Where converging, diverging, or the last-named tubes, are *unsubmerged*, *partially* submerged, or *wholly* submerged, the same theoretical formulæ are used as though the tube were of equal bore throughout, the coefficient only being altered to suit the case, and in converging and diverging tubes the area in square feet being taken at the narrowest portion; and it is of the greatest importance in determining the coefficient to be used, to observe accurately the *form* of the entrance to the orifice or tube and its approaches, when it is required to obtain the true from the theoretical discharge.

When a short tube is inclined obliquely (see Diagram 6, Plate III.) the coefficients of discharge decrease and vary at from 0·806 at 5° to 0·710 at 65° with the *horizontal*.

It must be borne in mind that in all short tubes, unless the tube be at least once and a half to twice the diameter, the coefficient will not be increased beyond that due to an orifice in a thin plate, the reason being that the contracted vein at less lengths does not fill the diameter of the pipe. If the tube is longer than about 2½ to 3 diameters, an extra loss occurs also from friction on the sides; this will be dealt with when we come to deal with *long* pipes under pressure.

CHAPTER IV.

PLATES IV. AND V.

ON THE FLOW OF WATER THROUGH LONG PIPES, AND THROUGH DISTRIBUTING PIPES IN TOWNS.

WHEN water flows through a series of pipes, three things require consideration: the head due to the velocity of entry, the head due to friction, and the head due to change of direction by bends; in long pipes, that is, pipes over 2000 diameters, the head due to velocity of entry may be neglected, for it bears such a slight proportion to the frictional head that it is of no practical importance, but in short pipes up to 2000 diameters it should never be neglected. The friction between the water and the sides of a pipe of the length l and diameter d is given by the formula

$$\frac{f \times l}{\frac{1}{4} \times D}; \qquad [50]$$

in which $\frac{1}{4}$ D equals the "hydraulic mean depth" or "hydraulic radius," which, in circular pipes, always equals one-fourth of the diameter, and f a coefficient, the value of which is given by Darcy as equal to

$$0{\cdot}005\left(1 + \frac{1}{48 \times \frac{1}{4} d}\right), \qquad [51]$$

where $\frac{1}{4} d$ is given in feet.

It is usual, however, in practice, to use a formula in which this friction is taken into account, and I have always used the following, deduced from Eytelwein, which are easily worked by logarithms, and give very accurate results, except with very low velocities, when a different formula should be used, which will be given hereafter:—

$$G = \sqrt{\frac{(3\,d)^5 \times H}{L}} \qquad [52]$$

$$H = \frac{G^2 \times L}{(3\,d)^5} \qquad [53]$$

$$d = \sqrt[5]{\left(\frac{G^2 \times L}{H}\right)} \div 3 \qquad [54]$$

$$L = \frac{(3\,d)^5 \times H}{G^2}; \qquad [55]$$

in which

G = gallons per minute;
H = head in feet;
d = diameter in inches;
L = length in yards.

These formulæ, which are suitable for a velocity of 2 to 3 feet per second, do *not* include the head due to the velocity of entry or bends, which in *all* cases of short pipes must be added to that necessary to overcome friction, as found by formulæ [52] to [55] inclusive, for obtaining the total head required for passing a given quantity of water.

The head due to the velocity of entry may be found by the formula [48], supposing that the orifice is of the form requiring a coefficient of 0·817, otherwise the formula [46] *reversed* must be used, applying the right coefficient.

Example.—What is the head required to force 100 gallons per minute through a 4-inch pipe 100 yards long?

		Head required, feet.
For velocity of entry ..	$\left(\frac{100}{4^2 \times 13}\right)^2 =$	0·23
For friction	$\frac{100^2 \times 100}{(3 \times 4)^5} =$	4·01
Total head required in feet ..		4·24

The *hydraulic inclination* of this pipe is therefore 4 ft. $0\frac{1}{8}$ in.

To make this clear, the *hydraulic inclination* of a pipe

being often mistaken for its actual fall from end to end, let us refer to Diagram 1, Plate IV.

The two pipes A B and C D of the same length have precisely the same *hydraulic inclination*, supposing the discharging ends B and D to be on the same level, notwithstanding the position of their entrances, A and C, in the vessel, or their actual inclinations.

If they be of the same diameter, and the orifices of entry similar, the velocities are also the same, and therefore the head h, which we will assume as the head due to velocity of entry, is similar; the head, therefore, to be divided by the length for the *hydraulic inclination* of both the pipes A B and C D is the total head H less h, the head required for velocity of entry, or $\frac{h_1}{l}$ is equal to the hydraulic inclinations of the pipes in question.

It is the practice of some engineers to obtain the head h by first obtaining an approximate velocity, and then, by a series of tentative operations, repeating the approximations until accuracy is obtained, but this, it appears to me, is at least a very clumsy way of attaining the desired end. The best plan is to assume any discharge at first, and calculate the heads required for velocity of entry, bends, and friction, and add these heads together; the proper discharge may then be found by the rule that in any pipe or series of pipes the diameter and length being constant the discharge varies directly as the square root of the head.

Example.—What is the discharge through a 6-inch pipe 3000 yards long with a head of 50 feet?

First, assume a discharge of 200 gallons per minute.

		Head required in feet.
For velocity of entry	$\left(\frac{200}{6^2 \times 13}\right)^2 =$	0·182
For friction	$\frac{200^2 \times 3000}{(3 \times 6)^5} =$	63·500
Total head in feet required per 200 gals. per minute		63·682

Working by logarithms :

$$\begin{array}{lr}
\text{Log } 200 = & 2\cdot30103 \\
 & 2\cdot67024 \\
\hline
 & -\ 1\cdot63079 \\
\times\ 2 & 2 \\
\hline
 & -\ 1\cdot26158 \\
\text{No. of log} = & \cdot182
\end{array}
\qquad
\begin{array}{lr}
\text{Log of } 6 = & 0\cdot77815 \\
\times\ 2 & 2 \\
\hline
 & 1\cdot55630 \\
\times\ 13 & 1\cdot11394 \\
\hline
 & 2\cdot67024
\end{array}$$

$$\begin{array}{lr}
\text{Log of } 18 = & 1\cdot25527 \\
 & 5 \\
\hline
 & 6\cdot27635
\end{array}
\qquad
\begin{array}{lr}
\text{Log of } 200 = & 2\cdot30103 \\
\times\ 2 & 2 \\
\hline
 & 4\cdot60206 \\
\times\ 3000 & 3\cdot47712 \\
\hline
 & 8\cdot07918 \\
\text{Log } 18^5 & 6\cdot27635 \\
\hline
 & 1\cdot80283 \\
\text{No. to log} & 63\cdot5
\end{array}$$

therefore the true discharge is equal to

Gallons per minute.

$$\frac{\sqrt{50} \times 200}{\sqrt{63\cdot682}} = 157\cdot03.$$

If bends had occurred in the pipe the head required for them would have been added before performing this last proportion.

For pipes with very low velocity it is best to use Prony's rule, which modified is

$$100 \times \sqrt{\text{H M D} \times \frac{\text{L}}{\text{H}}} - 0\cdot15, \qquad [56]$$

equal to velocity in feet per second, in which H M D = the hydraulic radius, or the area divided by the wetted perimeter,

both in feet, and $\frac{L}{H}$ the *hydraulic inclination* of the pipe in feet (see remarks on *hydraulic inclination*).

The rule may be altered to

$$G = \left\{\sqrt{\left(16 \cdot 353 \times \frac{H \times d}{L} + 0 \cdot 00665\right)} - 0 \cdot 0816\right\} \times d^2 \times 2 \cdot 04, \quad [57]$$

and to

$$H = \frac{\left\{\left(\frac{G}{2 \cdot 04 \times d^2} + 0 \cdot 0816\right)^2 - 0 \cdot 00665\right\} \times \frac{L}{d}}{16 \cdot 353}, \quad [58]$$

where the letters have the same signification as in formula [52].

Rules [57] and [58] only give the head required for friction, that for velocity of entry and bends must always be added.

There is yet another loss of head to be treated of, viz. that due to bends, the formulæ for which may be taken as follows:—

$$H = \left\{0 \cdot 131 + \left(1 \cdot 847 \times \left(\frac{r}{R}\right)^{3 \cdot 5}\right\} \times \frac{V^2 \times \phi}{960}; \quad [58A]$$

and

$$V^2 = \frac{960 \times H}{\phi \times \left\{0 \cdot 131 + \left(1 \cdot 847 \times \left(\frac{r}{R}\right)^{3 \cdot 5}\right\}}; \quad [58B]$$

in which

H = the head due to change of direction in inches;
r = radius of the bore of the pipe in inches;
R = radius of the centre line of the bend in inches;
ϕ = angle of bend in degrees;
V = velocity of discharge in feet per second.

As before stated with regard to $\frac{3}{2}$, the fraction $\frac{7}{2}$ or $3 \cdot 5$ means the 7th power of the square root of the number to which it is attached, or the logarithm of the number multi-

plied by $3 \cdot 5$ gives the logarithm of the number raised to the $3 \cdot 5$ power.

The preceding formulæ have reference chiefly to circular pipes, but the discharge of rectangular or any form of pipes is easily found by finding the *hydraulic* radius or hydraulic mean depth, equal to the area divided by the wetted perimeter for the form required, and then calculating the discharge of a pipe of the same hydraulic mean depth, which will equal the discharge required. This is founded on the rule that *the velocity of discharge, whatever may be the form of the pipe or channel, is proportional to the hydraulic mean depth.*

Before applying any of these formulæ, however, there are several most important subjects to consider.

A pipe may follow the section of the ground so long as it nowhere rise above its hydraulic mean gradient, or virtual declivity. Thus in Plate IV., Figure 2, the discharge due to the pipe A B, if of equal diameter throughout, is equal to the head H and the length A B, *h* being the head for velocity of entry.

If, however, the ground rises above this line of virtual declivity, as in Figure 3, Plate IV., the discharge of the pipe C D, if of equal diameter throughout, will only be equal to the head due to the difference of level of G and E, and the length C E, for the lower part of the pipe would only act as a trough, and convey the water due to the head H for friction, and *h* for velocity of entry.

The pipe from F to D would therefore be valueless for attaching services to, as the pressure would be lost if a full draught prevailed at D.

The proper way would be to use pipes of two diameters, the larger one from C to E, and the smaller one from E to D.

This shows how important it is in laying out compound mains to first calculate the hydraulic gradients, and then lay them on the plotted section of the ground, so as to see that none of the hill-tops rise above them.

When a compound main pipe is given, of which it is required to calculate the gradients, the best plan is to *assume*

any discharge, and calculate the head due to velocity of entry and friction and bends for each pipe on this *assumed* discharge; by this means a total head required for the *assumed* discharge will be obtained, and as the real head will be divided amongst the several pipes in the same proportion, the actual gradients can easily be calculated and laid on the section.

Before entering on the subject of the distribution of the water in the town after the supply has been brought there by the main pipe from the service reservoir, the following important laws with respect to pipes should be well considered. Rules [60], [62], and [63], refer only to long pipes, or pipes of 2000 diameters.

[59] When the *length* and *diameter* are *constant*, the discharge varies directly as the square root of the head. The converse of this is also true, viz. that the head is directly as the square of the discharge.

[60] When the *head* and *length* are *constant*, the discharge is directly as the 2·5 power of the diameter, and conversely the diameter is directly as the 2·5 root of the discharge.

[61] When the *discharge* and *diameter* are *constant*, the head is directly as the length.*

[62] When the *length* and *discharge* are *constant*, the head will be *inversely* as the 5th power of the diameter, and conversely the diameter will be *inversely* as the 5th root of the head.

[63] When the *head* and *diameter* are *constant*, the discharge will be *inversely* as the square root of the length, and conversely the length is *inversely* as the square of the discharge.

[64] If a pipe of *uniform* diameter has a series of branches diverging from it, so that the flow of water through it becomes less and less at an uniform rate until the pipe terminates at a dead end, the virtual declivity goes on diminishing, being proportional to the *square of the distance*

* The 2·5 power, or 5th power of square root, of any number is found by multiplying the logarithm of the number by 2·5, and finding the number corresponding to this resulting log. The 2·5 root, or square of the 5th root, is found by dividing the log. of the number by 2·5.

from the dead end, and the excess of the head at any point above the head at the dead end is proportional to the *cube of the distance from the dead end*, and the total virtual fall from the commencement of the pipe to the dead end is *one third* of what it would have been had the whole quantity of water flowed along the pipe without diverging into branch pipes.

[65] When a horizontal pipe A B (see Plate IV., Figure 4), and a head at A, is that due to a velocity in C D, the discharge from the pipe A B will be equal to that from C D, but a peculiar property belongs to the pipe A B in the position in which it is here placed, for if we cut it short at any point *e*, or lengthen it to any extent to E, the discharge will remain the same, and equal to that through the horizontal pipe C D. The velocity in A B (at the angle of inclination A B C, when A C equals the head for friction, or *f* in the diagram, and A B equals C D) is therefore such that it remains unaffected by the length A E, or A *e*, to which it may be extended or cut short, and at this inclination the water in the pipe A B is said to be "in train," and hence the utility of fixing a general *hydraulic inclination* for the distribution pipes in a town, to be described hereafter.

We will now proceed, with the aid of these rules, to describe the best mode of proceeding in order to determine the diameters of the distributing pipes in the town, and to do this we will refer to the diagram of a supposititious town water supply, and follow step by step the calculations necessary.

It is proposed to supply a town arranged as in the diagram Plate V. (inclusive of increase) with water. By what sized mains and distributing pipes will the proposed supply be effected, after leaving the service reservoir?

As to Quantity.—It is first necessary to estimate the quantity of water which will be required.*

We will assume that 40 gallons per head per day is this quantity; it of course varies in different localities—in England, from 25 to 30 gallons per head per day being generally

* This, in some cases, has been done to determine the drainage area required to the *storage* reservoir.

ample for all purposes. At this rate (40 gallons), a daily supply of 40 × 4000 equal to 160,000 gallons would be required in 24 hours.

It would, however, be exceedingly erroneous to calculate the sizes of the distribution mains on this assumption, for it is evident that nearly the whole of the supply for domestic purposes will be drawn off during a limited number of hours. Various estimates have been made of the most rapid draught which occurs, and it is evident that this varies much in different towns.

Mr. Hawksley, one of the best authorities on this subject, assumes that $\frac{1}{4}$ of the whole supply may be drawn off in 1 hour, other authorities have found that $\frac{1}{10}$ of the supply is drawn off in 1 hour, or the whole supply in 10 hours, and due regard must be had to the eventual requirements of a town which is likely to increase, in fixing the rate of most rapid draught. In the case before us we will assume, with Mr. Hawksley, that the whole supply will be drawn off in 4 hours, and therefore, in the case under our consideration, the supply for which the diameters of the mains must be calculated will be equal to 160,000 × 6 = at the rate of 960,000 gallons in 24 hours.

As to Head.—A proper rate of virtual head must be given to the pipes, and as a clear pressure of at least 20 feet over the tops of the houses should be left even during the most rapid flow of water, in order that the upper stories of the houses or places of business may be supplied, and that a sufficient jet may be obtained directly from the main in cases of fire, the general rate of virtual declivity to be fixed for the distributing pipes must depend on the contour levels of the town in question.

While on this subject, I would suggest that where it is proposed to supply a town with water, a plan showing its contour lines should be prepared; this is easily done, if none exists, by obtaining the sections of the streets, and from them laying down fixed levels on the plan—in fact, in all cases of water supply and drainage a plan of this description is invaluable, both for the first calculations of the sizes of the

distribution mains or sewers and for the laying of new mains rendered necessary by the want of increased distribution owing to an increase in population. It is also necessary in large towns to ascertain the proportional distribution of the inhabitants in the town, so as to determine the proportionate quantity of water required in each district. Returning to the example under consideration: It is usual, in the first place, to calculate a main from the *service* reservoir to the centre of the town of sufficient size to deliver the quantity required during the most rapid discharge estimated; this in our case is at the rate of 960,000 gallons in 24 hours, or say 667 gallons per minute. It is proposed to extend the supply to the contour line 110 feet below the *service* reservoir, and as it may be necessary to extend it eventually to the contour 90 feet below same, the available head at this point will be 90 feet, but as the water should issue at this point with a head of 20 feet over the tops of the houses in case of fire, we will assume the available head to be 50 feet, which head may be assumed as the virtual declivity of the pipes, and be taken in computing the size of the main from the *service* reservoir to the centre of the town, by Formula [54]; the pipe being a long one, no notice need be taken of the velocity of entry.

Diameter in inches.

$$\sqrt[5]{\frac{667^2 \times 1452}{50}} \div 3 \quad = \quad 8 \cdot 80$$

$$\begin{array}{rr}
\text{Log } 667 = & 2 \cdot 82413 \\
\times 2 & 2 \\
\hline
 & 5 \cdot 64826 \\
\text{Log } 1452 = & 3 \cdot 16196 \\
\hline
 & 8 \cdot 81022 \\
\text{Log } 50 = & 1 \cdot 69897 \\
\hline
\div 5 & 7 \cdot 11125 \\
\hline
 & 1 \cdot 42225
\end{array}$$

26·4, number corresponding to logarithm,

which divided by 3 is equal to 8·80, or say 9 inches, and adding 1 inch for possible corrosion, say 10 inches.

Having now determined the diameter of the main from the *service* reservoir to the centre of the town, and remembering that in some towns the estimate of the full quantity in 4 hours would be too much, 8 or even 10 hours may frequently be taken, we will now fix the sizes of the distribution mains on the same assumption, and allowing 4 feet loss of head for every 220 yards.

In adopting the loss of head equal to 4 feet in every 220 yards, it must be remembered that it is only an assumption, and it is *not* meant in assuming it that the *service* reservoir should be placed at a height above the town which would allow of this loss of head in all the mains *at one time*, for it is evident that this would in most cases be impossible, and it is also unnecessary, 90 to 150 feet above the highest portion of the town to be supplied being in all cases ample. The reason that it is best to act in *calculating* the sizes of the distribution pipes *on the assumption* that a loss of head of 4 feet in every 220 yards occurs, and that the whole of the water taken off from each length has to be passed to the end of that length, is that in some few mains this loss may occur, and they must be proportioned to the greatest hourly demand, even though this demand and consequent loss of head only occurred in one length of 220 yards for one second of time.

It will be evident that with the town under consideration, if *an actual loss of head of 4 feet in every 220 yards of main in the town occurred at one second of time*, and an allowance be made for discharge under a clear head of 20 feet over the tops of the houses in the highest portions, the houses being assumed 20 feet high, that the *service* reservoir should be 260 feet above the highest portions of the town it is only placed 90 feet above, which is of course ample, as hereafter shown, for the whole of the services could not be drawing at the *assumed* rate at one time. In the following calculations, therefore, it is only *assumed* in calculating the diameters of the different mains that a loss of head of 4 feet in every 220 yards will occur, the actual capability of the reticulation pipes with the *service* reservoir, at the height shown on the diagram, being afterwards determined by Rule [59].

This must be well understood, as errors are frequently made by taking a statement as an actual fact, whereas it is only an assumption for the purposes of calculation and for proportioning *each* pipe to the greatest hourly demand which it may possibly have to meet in a second of time, though of course all the pipes in the town would not be called upon to meet it at one time.

The part etched and marked *a b c d* in the diagram, Plate V., is supposed to contain 2000 inhabitants, and therefore requires, on the assumption before stated, a supply at the rate of

$$2000 \times 40 \times 6 = 480{,}000 \text{ gallons in 24 hours,}$$

or say,

$$334 \text{ gallons per minute.}$$

The remaining 2000 persons are distributed over the remainder of the town, requiring a supply at the rate of

$$2000 \times 40 \times 6 = 480{,}000 \text{ gallons in 24 hours,}$$

or say,

$$334 \text{ gallons per minute.}$$

In this town it will be best to first divide the supply amongst 3 principal mains running north and south, afterwards determining the diameters of the branches by Rule [60]; these mains should run up the streets U T, H Q, and K V. The main U T should be capable of supplying all the water required as far as the street G R, viz. 365,710 gallons in 24 hours, or 254 gallons per minute.

The first 220 yards from C to D will therefore require a diameter equal to the supply of 182,855 gallons in 24 hours, or 127 gallons per minute;* and the second 220 yards, 68,570 gallons in 24 hours, or 48 gallons per minute.

The diameters required are therefore as follows, by Rule [54]:—

* In all these cases it is assumed that the whole of the water taken off from each length has to be passed to the *end* of that length.

Diameter in inches.

For first 220 yards $\sqrt[5]{\frac{127^2 \times 220}{4}} \div 3 = 5 \cdot 16$, say 5 ins.

„ second „ $\sqrt[5]{\frac{48^2 \times 220}{4}} \div 3 = 3 \cdot 49$, say 4 ins.

And by the diagram, from C to B and B to I will be similar sizes.

We next come to the main to be laid up the street HQ.

This main should be capable of supplying all the water required as far as GR on the one side and JP on the other; for the first 220 yards, WX, a quantity equal to 228,570 gallons in 24 hours, or say 159 gallons per minute, is necessary, and for the next 220 yards, or WH, 68,570 gallons in 24 hours, or 48 gallons per minute, are required, therefore—

Diameter in inches.

For the diameter from X to W $\sqrt[5]{\frac{159^2 \times 220}{4}} \div 3 = 5 \cdot 64$ say 6 ins.

„ „ W to H $\sqrt[5]{\frac{48^2 \times 220}{4}} \div 3 = 3 \cdot 49$ say 4 ins.

And by the diagram from X to Y and Y to Q the diameters will also be 6 inches and 4 inches respectively.

We have now to consider the main up the street KV.

For the first 220 yards 68,570 gallons in 24 hours, or 48 gallons per minute, will be required; for the second 220 yards 34,285 gallons in 24 hours, or 24 gallons per minute. The diameters required are therefore as follows, by Rule [54]:—

Diameter in inches.

For first 220 yards $\sqrt[5]{\frac{48^2 \times 220}{4}} \div 3 = 3 \cdot 49$, say 4 ins.

„ second „ $\sqrt[5]{\frac{24^2 \times 220}{4}} \div 3 = 2 \cdot 65$, say 3 ins.

The main C M through the centre of the town should also have a practical taper.

For the first 220 yards it will be 10 inches, discharging

the full quantity of 667 gallons per minute, that is up to the street FS.

Diameter in inches.

Between F S and G R $\sqrt[5]{\frac{572^2 \times 220}{4}} \div 3 = 9 \cdot 4$

" G R " H Q $\sqrt[5]{\frac{413^2 \times 220}{4}} \div 3 = 8 \cdot 27$

" H Q " J P $\sqrt[5]{\frac{254^2 \times 220}{4}} \div 3 = 6 \cdot 8$

" J P " K V $\sqrt[5]{\frac{156^2 \times 220}{4}} \div 3 = 5 \cdot 6$

As the head and length used in the above calculations are *constant*, the diameters of the mains in any of the above streets might have been found by Rule [60] for example. What should the diameter of the main from W to X in street H Q be to supply 159 gallons per minute, the diameter of the main from C to D being 5·16 inches, discharging 127 gallons per minute?

Then as

$$\sqrt[2.5]{127} : \sqrt[2.5]{159} :: 5 \cdot 16 : 5 \cdot 64,$$

or the same as calculated before, and the practical diameter of the pipe may be taken as 6 inches.

The rule is worked as follows by logarithms:—

$$\begin{array}{rr} \text{Log of } 159 = & 2 \cdot 20140 \\ \div\, 2 \cdot 5 \;= & \overline{0 \cdot 88056} \\ \times\, 5 \cdot 16 = & 0 \cdot 71264 \\ & \overline{1 \cdot 59320} \\ \div \sqrt[2.5]{129} & 0 \cdot 84152 \\ & \overline{0 \cdot 75168} \end{array}$$

Number corresponding to log 5·64.

Having now determined the diameters of the principal mains in the town, it will be necessary to find the diameters of the branch mains, so that services may be attached for the supply of the houses.

For the cross streets in the most thickly populated part we shall require in each 220 yards a quantity of water equal to 71,712 gallons in 24 hours, or 49·8 gallons per minute. The diameters of these cross branch mains may then be determined from any one of the principal mains by Rule [60]; say we determine them from C D.

As

Diameter of Main. Inches.

$$\sqrt[2\cdot5]{127} : \sqrt[2\cdot5]{49\cdot8} :: 5\cdot16 : 3\cdot49, \text{ say 4 inches.}$$

By logarithms :—

$$\begin{array}{rl}
\text{Log of } 49\cdot8 = & 1\cdot69723 \\ \hline
\div\, 2\cdot5 = & 0\cdot67889 \\
\times\, 5\cdot16 = & 0\cdot69897 \\ \hline
 & 1\cdot37786 \\
\div \text{ log of } \sqrt[2\cdot5]{127} & 0\cdot84152 \\ \hline
 & 0\cdot53634
\end{array}$$

Number corresponding to log 3·49.

$$\begin{array}{rl}
\text{Log of } 127 = & 2\cdot10380 \\
\div\, 2\cdot5 = & 0\cdot84152.
\end{array}$$

And for the cross streets in the other portion of the town to deliver a quantity equal to 34,285 gallons in 24 hours, or say 24 gallons per minute in every 220 yards.

As

Diameter in inches.

$$\sqrt[2\cdot5]{127} : \sqrt[2\cdot5]{24} :: 5\cdot16 : 2\cdot57, \text{ say 3 inches.}$$

By logarithms :—

$$\begin{array}{rl}
 & 1\cdot38021 \\ \hline
\div\, 2\cdot5 = & 0\cdot55208 \\
\text{Log } 5\cdot16 & 0\cdot71264 \\ \hline
 & 1\cdot26472 \\
\div \text{ log of } \sqrt[2\cdot5]{127} & 0\cdot84152 \\ \hline
 & 0\cdot42320
\end{array}$$

Number corresponding to log 2·65.

From these calculations and the diagram it is evident, therefore, that the streets F S, G R, J P should be laid with 4-inch pipes through the most populous parts, and 3-inch for the other parts.

With regard to the leading main or supply conduit from the *storage* reservoir to the service reservoir, it should always be large enough to convey the full quantity of water which it is estimated can be derived from the drainage area, rendering necessary an extension only of reservoir room when the population requires an increased supply, a much less costly work generally than a duplication of the pipe lines.

To check the former computations to see that the *theoretical* dimensions of the main are large enough for the passage of the water required, we may reverse the previous calculations, and estimate the loss of head occasioned by each branch-pipe and the main pipe.

For calculating the loss of head in the street mains, it is best to first find the proportional discharge of each pipe, the 10-inch main (theoretical diameter 10 inches) being assumed at 667 gallons, or the total quantity for the first 220 yards above the 5-inch main.

On this assumption, therefore, and taking the *theoretical* dimensions of the mains, the 10-inch main will discharge 667 gallons per minute at street C D. The discharge of the 5-inch main will therefore be

$$\text{as } 10^{2\cdot5} : 5\cdot16^{2\cdot5} :: 667 : 127 \text{ gallons per minute.}$$

The discharge will therefore be, for the portion of 10-inch main between U T and F S, taking at first only 220 yards of main to facilitate calculation, 540 gallons per minute.*

* Only 220 yards of each main is at first dealt with, so that Rule [60] may be applied, for this rule is only applicable when the *head* and *length* are constant. The total heads required are found afterwards by proportion, using Rule [61], as will be seen hereafter; the quantities 540, 501, 459, &c., are, therefore, only *assumed* quantities for the purpose of continuing the calculation.

For the next 220 yards the quantity to be deducted will be

as $10^{2\cdot5} : 3\cdot49^{2\cdot5} :: 540 : 39$ gallons per minute.

Therefore for the space between F S and G R of the 9-inch main the discharge will be 501 gallons per minute.

For the next 220 yards the quantity to be deducted will be

as $9\cdot4^{2\cdot5} : 3\cdot49^{2\cdot5} :: 501 : 42$ gallons per minute.

Therefore for the space between G R and H Q, or the 8-inch main, the discharge will be 459 gallons per minute.

For the next 220 yards the quantity to be deducted will be

as $8\cdot27^{2\cdot5} : 5\cdot64^{2\cdot5} :: 459 : 176$ gallons per minute.

Therefore for the space between H Q and J P along the 7-inch main a quantity (assumed) of 283 gallons per minute will pass.

This will be again reduced at cross street J P by

as $6\cdot8^{2\cdot5} : 3\cdot49^{2\cdot5} :: 283 : 53$ gallons per minute.

The quantity passing along the 6-inch main will therefore be 230 gallons per minute.

This will be again reduced at cross street K V to

as $5\cdot6^{2\cdot5} : 3\cdot49^{2\cdot5} :: 230 : 70$ gallons per minute.

We have now to find the proportional discharges of 220 yards of the 3-inch and 4-inch mains in the upper part of the town not yet done.

						Gals. per min.
For 220 yds.	4-in.	in street	U T,	as $5\cdot16^{2\cdot5} : 3\cdot49^{2\cdot5}$	::	79 : 30
,,	3-in.	,,	F S,	as $3\cdot49^{2\cdot5} : 2\cdot65^{2\cdot5}$	::	39 : 19
,,	3-in.	,,	G R,	as $3\cdot49^{2\cdot5} : 2\cdot65^{2\cdot5}$	::	42 : 21
,,	4-in.	,,	H Q,	as $5\cdot64^{2\cdot5} : 3\cdot49^{2\cdot5}$	::	176 : 53
,,	3-in.	,,	J P,	as $3\cdot49^{2\cdot5} : 2\cdot65^{2\cdot5}$	::	53 : 27
,,	3-in.	,,	K V,	as $3\cdot49^{2\cdot5} : 2\cdot65^{2\cdot5}$	::	70 : 35
,,	4-in.	,,	D L,	as $5\cdot16^{2\cdot5} : 3\cdot49^{2\cdot5}$	::	127 : 48
,,	3-in.	,,	E I,	as $3\cdot49^{2\cdot5} : 2\cdot65^{2\cdot5}$	::	30 : 15

From these calculations we shall be able to compute the probable comparative heads lost in all the mains for 220 yards of those mains, and so the total head by Rule [61].

The head required for the 10-inch main from the *service* reservoir to the point C on plan will be for the discharge of 667 gallons per minute, taking only the theoretical diameter.

Head required.
Feet.

$$\frac{667^2 \times 880}{(3 \times 10)^5} = 16 \cdot 11.$$

We will now find out the head required for each 220 yards of the different mains for the proportional discharges before calculated, and assuming the 220 yards of 10-inch main as the standard discharging 667 gallons per minute for the purposes of the calculation.

				Head required for 220 yards. Feet.
For 5 in.	theoretical	diameter	5·16 in.	$\frac{127^2 \times 220}{(3 \times 5 \cdot 16)^5} = 3 \cdot 99$
„ 4 in.	„	„	3·49 in.	$\frac{39^2 \times 220}{(3 \times 3 \cdot 49)^5} = 2 \cdot 66$
„ 4 in.	„	„	3·49 in.	$\frac{42^2 \times 220}{(3 \times 3 \cdot 49)^5} = 3 \cdot 08$
„ 6 in.	„	„	5·64 in.	$\frac{176^2 \times 220}{(3 \times 5 \cdot 64)^5} = 4 \cdot 91$
„ 4 in.	„	„	3·49 in.	$\frac{53^2 \times 220}{(3 \times 3 \cdot 49)^5} = 4 \cdot 91$
„ 4 in.	„	„	3·49 in.	$\frac{70^2 \times 220}{(3 \times 3 \cdot 49)^5} = 8 \cdot 57$
„ 10 in.	„	„	10 in.	$\frac{540^2 \times 220}{(3 \times 10)^5} = 2 \cdot 64$
„ 9 in.	„	„	9·4 in.	$\frac{501^2 \times 220}{(3 \times 9 \cdot 4)^5} = 3 \cdot 09$

				Head required for 220 yards. Feet.
For 8 in. theoretical diameter		8·27 in.	$\frac{459^2 \times 220}{(3 \times 8{\cdot}27)^5}$	= 4·93
„ 7 in.	„ „	6·8 in.	$\frac{283^2 \times 220}{(3 \times 6{\cdot}8)^5}$	= 4·98
„ 6 in.	„ „	5·6 in.	$\frac{230^2 \times 220}{(3 \times 5{\cdot}6)^5}$	= 8·69
Street.				
U T 4 in.	„ „	3·49 in.	$\frac{30^2 \times 220}{(3 \times 3{\cdot}49)^5}$	= 1·57
F S 3 in.	„ „	2·65 in.	$\frac{19^2 \times 220}{(3 \times 2{\cdot}65)^5}$	= 2·50
G R 3 in.	„ „	2·65 in.	$\frac{21^2 \times 220}{(3 \times 2{\cdot}65)^5}$	= 3·05
H Q 4 in.	„ „	3·49 in.	$\frac{53^2 \times 220}{(3 \times 3{\cdot}49)^5}$	= 4·91
J P 3 in.	„ „	2·65 in.	$\frac{27^2 \times 220}{(3 \times 2{\cdot}65)^5}$	= 5·05
K V 3 in.	„ „	2·65 in.	$\frac{35^2 \times 220}{(3 \times 2{\cdot}65)^5}$	= 8·48
D L 4 in.	„ „	3·49 in.	$\frac{48^2 \times 220}{(3 \times 3{\cdot}49)^5}$	= 4·03
E I 3 in.	„ „	2·65 in.	$\frac{15^2 \times 220}{(3 \times 2{\cdot}65)^5}$	= 1·56

From these calculations we can now determine the head lost in all the mains, on the same assumption of 667 gallons per minute being discharged, and if our calculations have been done correctly, this should agree with the allowance *assumed* (see pages 35 to 39), being 4 feet in every 220 yards, plus the head required for the passage of 667

gallons per minute through the 10-inch main from the *service* reservoir to the street U T, viz. 10,780 yards, divided by 220 and multiplied by 4, or equal to 196 feet, and adding 16·11, the head required for the 10-inch main to street U T, brings up a *total* of 212·11 *feet*, with which the following calculations of *total head* required should agree.

The heads are calculated by Rule [61].

Street.					Total head required. Feet.
U T	for	5-inch main, as	220 : 440 :: 3·99 :		7·98
F S	„	4 „	220 : 440 :: 2·66 :		5·32
G R	„	4 „	220 : 440 :: 3·08 :		6·16
H Q	„	6 „	220 : 440 :: 4·91 :		9·82
J P	„	4 „	220 : 440 :: 4·91 :		9·82
K V	„	4 „	220 : 440 :: 8·57 :		17·14
C M	„	10 „	220 : 220 :: 2·64 :		2·64
C M	„	9 „	220 : 220 :: 3·09 :		3·09
C M	„	8 „	220 : 220 :: 4·93 :		4·93
C M	„	7 „	220 : 220 :: 4·98 :		4·98
C M	„	6 „	220 : 220 :: 8·69 :		8·69
U T	„	4 „	220 : 440 :: 1·57 :		3·14
F S	„	3 „	220 : 440 :: 2·50 :		5·00
G R	„	3 „	220 : 440 :: 3·05 :		6·10
H Q	„	4 „	220 : 440 :: 4·91 :		9·82
J P	„	3 „	220 : 440 :: 5·05 :		10·10
K V	„	3 „	220 : 440 :: 8·48 :		16·96
D L and B N	„	4 „	220 : 2200 :: 4·03 :		40·30
E I and A O	„	3 „	220 : 2200 :: 1·56 :		15·60
					187·59
Add 4 feet for passing 667 gallons per minute to street F S (see page 40)					4·00
Add 16·11 feet per head required in 10-inch main					16·11
Total head in feet and decimals					207·70

Or a difference of only 4·41, or 2¼ per cent., from the total head assumed in calculating the distribution pipes. We are

now, therefore, enabled by Rule [59] to calculate the actual demand which, if made on the town at one second of time, still assuming the theoretical diameters, would give a clear head of 20 feet over the tops of the houses at the highest point in the district under consideration, viz. the 90 feet contour line, as follows:—

	Gallons per minute.		Gallons per minute.
As $\sqrt{207 \cdot 70} : \sqrt{50} ::$	667	:	327.

The reticulation of this town may therefore be considered sufficient for a supply *at the rate of*, with the theoretical diameters, $327 \times 60 \times 24 = 470{,}880$ gallons in 24 hours, with the service reservoir at its present level, or equal to the improbable contingency of the *whole* population drawing off the total supply of 40 gallons per head continuously in about 9 hours, and still affording a pressure, in case of fire, of 20 feet above the tops of the houses, and equal also to a possible emergency of 1760 yards of houses drawing off the whole of their *proportionate supply* in four hours.

Having now gone into this subject as fully and as accurately as the circumstances of the case will allow, a few general remarks may be useful to the reader.

1st. Where the town is very irregular in levels, or many suburbs lying at some distance from it have to be supplied, and where, therefore, an excessive pressure would be given in the lower portions of the town by placing the *service* reservoir at a sufficient height above the higher portions, it is best to supply the town under different zones of pressure, as is to my own knowledge done in Manchester and Halifax, England. Where also, from the large increase of a town population, the higher parts do not obtain a sufficient pressure, a series of service reservoirs may be constructed for each district; these are then filled in the night, and in the day the head, which would otherwise be lost in passing the water to the higher districts through the branch mains and other reticulation, is gained for the supply of the districts

in question; each case, of course, varies, and demands its own special considerations; but the principle, such as I have laid down for determining the sizes of the distribution mains, remains the same.

2nd. The position of valves, fire-cocks, &c., are not shown in the diagram, it would be foreign to the object of this work to so show them; the valves should be placed so that as little of the town supply should be shut off as possible during the attaching of services or the occasional necessary repairs required.

3rd. In designing the mains or conduits for the supply, a special difference always exists in the supply main *from* the *storage* reservoir to the *service* reservoir, and the supply main *from* the *service* reservoir to the town. The first named has to supply a *constant* quantity in the case before us of 160,000 gallons in 24 hours. The second named has to supply a *variable* quantity and must be proportionate to the greatest estimated hourly draught, or in our case *at the rate of* 960,000 gallons in 24 hours.

4th. The larger the *service* reservoir the better, of course, three or four days supply at the *greatest estimated hourly demand* is usually considered sufficient to allow of any stoppage between the storage and service reservoirs.

5th. The distribution pipes should always be connected with the main supply from the *storage* reservoir in case of repairs being required to the *service* reservoir, or in case of cleansing, &c.

6th. The main object of a *service* reservoir is to equalise the distribution and *reduce* the supply main from the storage reservoir to a minimum (see 3rd remark).

7th. All the pipes should be connected with each other in the system of distribution, so that the water may have a free flow, and as few dead ends put in as possible; in fact, where it is necessary to put an end to the pipe, a branch or turn-up fire-cock should be placed so that the mains may be scoured occasionally.

Note referred to on page 42.—The *proportional* discharges

vary from those for which the diameters of the mains were calculated, owing to the assumed *constant* flow of water of 667 gallons per minute being distributed amongst them according to their diameters. The check of the whole work is that the *total head* calculated from these *proportional* discharges agrees to $2\frac{1}{4}$ per cent. with the total head required, by allowing 4 feet loss of head in every 220 yards of street main in the town in question.

It may be remarked that the *proportional* quantity which would flow down the actual *practical* mains laid may be easily obtained from the *proportional* quantities previously given for the theoretical mains by Rule [60]; for instance, supposing we require to know the actual *proportional* discharge at the end of each 220 yards of the 6-inch main in street H Q, the proportional discharge through the theoretical diameter being 176 gallons per minute, we have

$$5 \cdot 64^{2 \cdot 5} : 6^{2 \cdot 5} :: 176 : 205 \text{ gallons per minute.}$$

The discharge of the 6-inch pipe is therefore 205 gallons per minute.

To those who are unacquainted with the use of logarithms, which are necessary to work all hydraulic calculations easily, I would recommend 'Chambers' Mathematical Tables'; they should also take one or two lessons from one competent to teach, and this will probably be found sufficient.

On the Flow of Water through Siphon Pipes.

Plate VI.

The siphon is illustrated on Plate VI., Diagram (1), and its action may be thus explained. It has been found that a column of water 33 or 34 feet high in a hollow tube, wherein a vacuum has been formed, can be supported by the pressure of the atmosphere upon a water surface, and if we suppose that at the highest portion of the siphon *c*, Diagram (1), a partition *d* is placed, the column between *c* and A B will not only be supported, but it will be forced against the partition *d* with a force equal to the weight of a similar column having

a base *d* and a height of from 33 to 34 feet, less the difference of level between the higher water surface A B and the partition *d*.

But if the siphon be filled with a fluid, the opposite side of the partition *d* is acted upon by a column with the base *d* and a height equal to 33 or 34 feet, less the difference of level between *d* and the surface of the water D E. It is evident, therefore, that this last pressure is less than the first, and the consequence is, that if no partition existed, as none does exist in the siphon, the liquid section at the highest portion of the siphon would be forced forward in the direction F C D by a force equal to the difference of the levels of the water surfaces A B and D E.

The head, therefore, to be taken in calculating the discharge of a siphon when uncovered at the low end is the difference of level between the surface of the water at its inlet and the centre of the end of the long leg, or difference between A B and F, and when both ends are submerged the difference between the upper and lower waters, or difference between A B and D E, and it is necessary in large siphons to have the lower leg immersed to prevent the accumulation of air in the lower leg, which would cut off the column.

For calculating the discharge through a siphon pipe, three things must be considered: the head due to velocity of entry, the head due to friction, and the head due to the bend at its summit; and the best plan is, when the diameter, length, and radius of bend and head are given, to calculate the heads required on an *assumed* discharge for velocity of entry, bends, and friction, and add them together, afterwards finding the true discharge by Rule [59] (see example in Appendix of examples to this work).

When the length, head, discharge, and radius of bend are given, and the diameter is required, *assume* any diameter and calculate the heads required with this diameter, and the known discharge and length, for velocity of entry, bends, and friction; then by Rule [62] find out what the diameter should be with the true head (see example in Appendix).

For *inverted* siphons both ends are usually submerged, and the head is the difference of level between the upper and lower waters. Rules [52] to [55] inclusive are used for friction, Rules [47] to [49] for velocity of entry if a coefficient of 0·817 is required, if not [46] reversed must be used, or

$$\left\{\left(\frac{D}{A} \div c\right) \div 8 \cdot 025\right\}^2 = H; \qquad [66]$$

in which

D = discharge in cubic feet per second;
A = area in square feet;
c = coefficient according to circumstances;
H = head in feet.

For bends use Rules [58A] and [58B].

The same rules may be used for siphons *proper*.

Square siphon pipes, often used for drainage, &c., can be assimilated to round ones (see page 32).

It may be mentioned that the practical depth to which a siphon *proper* should be designed to draw should seldom exceed 25 or 26 feet.

CHAPTER V.

ON THE FLOW OF WATER FROM JETS, AND THE HEIGHTS ATTAINED BY DIFFERENT DIAMETERS OF JET.

ON this subject there are few experiments. From the best we have it would appear that, although theoretically the jet should rise to a height H, owing to the resistance of the air it only attains a height H^1, and that the *difference*, or H^2, increases with the absolute height of the jet and diminishes with an increase in the diameter. It is also found that H^2 increases nearly in the ratio of the *square of the head*, and that the curve obtained from different heights of jet having the same diameter approaches nearly to that of a parabola, and that the *head* being *constant*, H^2 varies in *inverse ratio* to the *diameter* of the jet.

From these observed facts, therefore, we may obtain a formula which, from the experiments at hand, is the nearest approximation obtainable, which is as follows:—

$$H^1 = \frac{H}{d} \times \cdot 0124; \qquad [67]$$

in which

H = the head on the jet in feet;
H^1 = the difference between the height of head and height of jet;
d = the diameter of the jet in $\frac{1}{8}$ths of an inch.

The discharge of jets of course varies with the form of the nozzle, and may be calculated by the formula

$$G = \sqrt{H} \times d^2 \times 0 \cdot 22 \qquad [68]$$

for a nozzle of a good form where the coefficient of discharge

is equal to 0·943 of the theoretical discharge, and the coefficient 0·22 in the above formula must be altered to suit the case. Some nozzles require a coefficient of only 0·18. In the above rule

H = the head of water in feet on the jet;
d = the diameter in $\frac{1}{8}$ths of an inch;
G = gallons discharged per minute.

When a jet occurs at the end of a long main, the head for passing the water through that main must be deducted, and the actual nett head on the jet taken. The discharge may in the first case be *assumed* as before shown, and the true result obtained by Rules [59], &c.; and as this system of calculation has been so often shown and referred to, it is unnecessary to here repeat it.

There are many interesting theories with regard to the paths of jets from oblique and straight nozzles, but as they are not of much value in practice, I omit them.

If mains are laid in a town, it may be mentioned that it would not be necessary then to calculate the head lost up to the point where the jet is wished to be played, as an application of the *pressure gauge* to the main during the hours of *greatest draught* would give the working pressure available for the playing of the jet.

CHAPTER VI.

PLATE VII.

ON THE FLOW OF WATER THROUGH CANALS, RIVERS, AQUEDUCTS, ETC.

Long Channels.—The theoretical velocity of water flowing through canals, &c., may be found by the formula,

$$\sqrt{\text{H M D} \times \text{F}} = \text{V} \qquad [69]$$

and the actual discharge by

$$\text{D} = \sqrt{\text{H M D} \times \text{F}} \times c \times \text{A}; \qquad [70]$$

in which

D = discharge in cubic feet per second;
V = velocity in feet per second;
H M D = the hydraulic mean depth or the area divided by the wetted perimeter;
F = the fall in 2 miles in feet;
c = a coefficient, according to circumstances;
A = the area in square feet.

Formula [70] is only applicable to *long* channels.

With regard to the coefficient c for large channels unencumbered with the growth of aquatic plants, and regular in their form, it may be taken as 0·84; for smaller channels in the same condition, as 0·75; but where the growth of aquatic plants is great, M. Girard conceives it necessary to multiply the wetted perimeter by 1·7 before dividing it into the area in order to obtain the hydraulic mean depth, so that for channels where the growth of these plants is great the hydraulic mean depth must be found as follows:—

$$\text{H M D} = \frac{\text{A}}{\text{P} \times 1{\cdot}7}; \qquad [71]$$

in which

A = the area in square feet;
P = the perimeter, or *wetted* border, in feet.

Short Channels.—Nothing, however, could be more erroneous than to use the formula where the channel is short, for instance, for a short culvert under a road, or for a flume across a stream, for then the head for velocity of entry must be considered. The best way in the case of a short flume or culvert, is to calculate the head for velocity of entry and friction separately for an *assumed* discharge, and add them together, then the true discharge may found by Rule [59].

For example.—What head must be given to a *short* aqueduct or culvert flume 50 feet long and 8 square feet area, and perimeter 8 lineal feet, to discharge 40 cubic feet per second, coefficient 0·75?

For velocity of entry.

$$\text{By rule [66]} \left\{\left(\frac{40}{8} \div 0 \cdot 75\right) \div 8 \cdot 025\right\}^2 = \overset{\text{Head in feet.}}{0 \cdot 67568.}$$

For friction, formula [70] reversed can be used, or

$$\left(\frac{D}{A} \div c\right)^2 \div \text{H M D} = \text{F}; \qquad [72]$$

∴ in our case

$$\left(\frac{40}{8} \div 0 \cdot 75\right)^2 \div 1 = \overset{\text{Fall in 2 miles.}}{43 \cdot 54.}$$

The fall, therefore, in 50 feet will be

$$\text{As } 10560 : 50 :: 43 \cdot 54 : \overset{\text{Fall in 50 feet.}}{0 \cdot 20615}$$

Add head for velocity of entry before obtained = 0·67568

Total fall required in flume = 0·88183

If the discharge had been *required* for the above flume, the head being given, it must be calculated as follows:—

Assume a discharge of say 32 cubic feet per second.

For velocity of entry.

$$\left\{\left(\frac{32}{8} \div 0 \cdot 75\right) \div 8 \cdot 025\right\}^2 = \overset{\text{Head in feet.}}{0 \cdot 4417.}$$

For friction.

$$\left(\frac{32}{8} \div 0\cdot75\right)^2 \div 1 = \overset{\text{Fall in 2 miles.}}{28\cdot4441}$$

Then

$$\text{As } 10560 : 50 :: 28\cdot4441 : \overset{\text{Fall in 50 feet.}}{0\cdot1346}$$

Add head for velocity of entry before obtained $\quad 0\cdot4417$

Total head required for 32 cube feet per second $= 0\cdot5763$

The actual given head being $0\cdot88183$ feet, we have by Rule [59]

$$\sqrt{0\cdot5763} : \sqrt{0\cdot88183} :: 32 : \begin{array}{c}\text{Actual discharge in}\\ \text{cube feet per second.}\end{array}$$

$$= \frac{0\cdot939 \times 32}{0\cdot7591} = 39\cdot61$$

as against 40 cube feet per second, the difference arising from not carrying out the decimal places far enough.

If the given head of $0\cdot88183$ had been taken, and the discharge taken by formula [70], we should have had

$$\sqrt{1 \times 186\cdot24} \times 0\cdot75 \times 8 = \overset{\begin{array}{c}\text{Discharge in cube}\\ \text{feet per second.}\end{array}}{81\cdot84,}$$

or more than double the true discharge, by not allowing for the velocity of entry.

This should be studied well, as mistakes are frequently made, for it will be seen that the head in the case cited to produce velocity of entry is actually greater than that required for friction, and therefore the head for velocity of entry should always be calculated until the length of the channel becomes so great that it bears such a slight proportion to the frictional head that it may be neglected; even *long channels* or aqueducts should be made rather wider at their entrances, and approach the "Vena Contracta" form as nearly as possible, to allow for the slight head required for velocity of entry.

On Contractions in River Channels.—When the banks of a river whose bed has a uniform inclination approach each other and contract the width of the channel in any way, as in Figs. 1 and 2, Plate VII., the water will rise in the channel above the contracted portion A, until the increased velocity of discharge compensates for the reduced cross section.

If we put d_1 for the increase of depth immediately above the contracted width in feet, and d_2 for the previous depth of the channel in feet, we shall find the quantity of water passing through the lower depth d_2 equal to

$$c \times l \times d_2 \times 8 \cdot 025 \sqrt{d_1}; \qquad [73]$$

in which l is the width of the contracted portion at A, and c a coefficient according to circumstances, and the quantity of water overflowing through d_1 equal to

$$\tfrac{2}{3} \times c \times b \times d_1 \times \sqrt{d_1} \times 8 \cdot 025; \qquad [74]$$

and hence the whole discharge through A is equal to

$$D = c \times l \times 8 \cdot 025 \sqrt{d_1} \times (d_2 + \tfrac{2}{3} \times d_1); \quad [75]$$

in which

D = the discharge in cubic feet per second;
c = a coefficient according to circumstances;
l = the width of the contracted portion at A in feet;
d_1 and d_2 as shown on diagram and before stated.

When the object is to find the width l of the contracted channel so that the depth of water in the upper reach shall be increased by a given depth d_1, we shall find

$$l = \frac{D}{c \times 8 \cdot 025 \sqrt{d_1} \times (d_2 + \tfrac{2}{3} \times d_1)}; \qquad [76]$$

signification of letters as in Rule [75].

On Velocity of Approach.

When the velocity of approach is considerable, or when the height h due to it becomes a large portion of d_1 its effect

must not be neglected. In this case, as before, we find the discharge through the depth d_2 equal to

$$D = c \times l \times d_2 \times 8 \cdot 025 \times \sqrt{(d_1 + h)} \qquad [77]$$

and the discharge through the depth d_1 equal to

$$D = \tfrac{2}{3} \times c \times l \times 8 \cdot 025 \times \{ (d_1 + h)^{1 \cdot 5} - h^{1 \cdot 5} \}, \qquad [78]$$

and hence the whole discharge is

$$D = c \times l \times 8 \cdot 025 \times \{ d_2 \times \sqrt{(d_1 + h)} + \tfrac{2}{3} \times [(d_1 + h)^{1 \cdot 5} - h^{1 \cdot 5}] \}, \qquad [79]$$

from which we may obtain

$$l = \frac{D}{c \times 8 \cdot 025 \times \{ d_2 \times \sqrt{(d_1 + h)} + \tfrac{2}{3} \times [(d_1 + h)^{1 \cdot 5} - h^{1 \cdot 5}] \}}; \qquad [80]$$

in which h equals the head in feet due to velocity of approach, and the other letters have the same signification as in Formula [75]. If the projecting spur at A be itself submerged, these formulæ can be extended by finding the discharges of the different sections according to the formula given and adding them together; this is so simple, and the resulting formula so lengthy to commit to paper, that they are omitted.

These formulæ are also applicable to cases of contraction of river channels caused by the construction of bridge piers and abutments, when the width l is put for the sum of the openings between them. The value of the coefficient will depend on the circumstances of the case.

For piers square to the channel take	0·6
When the angles of the cutwaters are obtuse take	0·7
And when curved and acute take	0·8

APPENDIX I.

PLATES VIII. AND IX.

THE water supplied to miners from the races of the Victoria Government is given in three ways:—

1st. Through a gauge box, with a pipe of a length equal to two diameters attached, as shown in the Diagrams, No. 1, 2, 3, Plate VIII.

2nd. Through a siphon pipe from the channel, as illustrated in Diagrams 4, 5, Plate VIII.

3rd. Through a sluice-gate opening of known dimensions, and with a stated head kept on it, Diagrams 1, 2, 3, Plate IX. The modes of calculating the quantity of water passed for these three different methods will now be dealt with in as simple a manner as the subject admits of.

For calculating the quantity of water passed through a short pipe of two diameters, with square edges, attached to a box, as shown in the Diagrams 1, 2, 3, Plate VIII., the following formula may be used:—

$$G = \sqrt{H} \times d^2 \times 13,$$

which, when the diameter is required, may be stated

$$d = \sqrt{\left(\frac{G}{\sqrt{H} \times 13}\right)}$$

and when the head is required

$$H = \left(\frac{G}{d^2 \times 13}\right)^2,$$

where

G = gallons per minute;
H = head in feet;
d = diameter in inches.

The Table, No. VII., is calculated from this rule, and can be extended as follows, this extension being based on one of the laws which govern pipes (see Chapter IV. of this work).

Rule.—The discharge of any pipe or series of pipes is proportional to the square root of the head when the length and diameter are constant; therefore, supposing we wanted the discharge through a 4-inch pipe with 4 feet head, we have by the table for one [1] foot head 208 gallons per minute; therefore, for 4 feet head the discharge will be

As $\sqrt{1} : \sqrt{4} :: 208 : 416$ gallons per minute.

By logarithms.

$$\text{Log of } 4 = 0\cdot6020600$$

$$\text{Log. of } 1 = 0\cdot0000000 \div 2 \quad 0\cdot3010300$$

$$\div 2 = 0\cdot0000000 \text{ log of } 208 = 2\cdot3180633$$

$$2\cdot6190933$$

$$\text{Log } \sqrt{1} = 0\cdot0000000$$

$$2\cdot6190933$$

Number corresponding to log 416

2. In calculating the quantity of water passed through a siphon pipe similar to the one shown in Plate VIII., Diagrams 4, 5, three things should actually enter into the calculation :—

1st. The head due to velocity of entry.
2nd. The head due to the bend at the top.
3rd. The head due to friction in the pipe.

But as the formula for obtaining the second is very complicated, and as if a large radius is given to the bend, it does not make much difference in the practical result, it may be omitted by the practical miner.

For the first head we shall have, therefore,

$$H = \left(\frac{G}{d^2 \times 13}\right)^2$$

and for the third

$$H = \frac{G^2 \times L}{(3 \times d)^5}.$$

Taking the siphon as shown in Diagrams 4, 5, Plate VIII., we will assume at first that the discharge will be 100 gallons per minute; therefore, neglecting the bend,

For velocity of entry.

		Head in feet required.
$\left(\frac{100}{2 \cdot 75^2 \times 13}\right)^2$	=	0·01034

For friction.

$\frac{100^2 \times 22}{(3 \times 2 \cdot 75)^5}$	=	5·75640
Total head	..	5·76674

By logarithms.

Log of 100 =	2·0000000	Log of 2·75 =	0·4393327
Log of $2 \cdot 75^2 \times 13$ =	1·9926088	× 2	2
	0·0073912		0·8786654
× 2	2	× 13	1·1139434
	0·0147824		1·9926088
Number corresponding	·01034		
Log of 100 =	2·0000000	Log of 2·75 =	0·4393327
× 2	2	" " 3 =	0·4771213
	4·0000000		0·9164540
× 22	1·3424227	× 5	5
	5·3424227		4·5822700
Log of $(3 \times 2 \cdot 75)^5$ =	4·5822700		
	0·7601527		
Number corresponding	5·7564		

Then by the rule before mentioned we have

Gallons per minute required.

$$\text{As } \sqrt{5 \cdot 76674} : \sqrt{10} :: 100 : 131 \cdot 68$$

By logarithms.

		Log of 10 =	1·0000000
		÷ 2	0·5000000
		× 100	2·0000000
Log of 5·76674 =	0·7609274		2·5000000
			0·3804637
For square root ÷ 2 =	0·3804637		2·1195363
		Number corresponding to log	131·68

The discharge through the siphon pipe is therefore, neglecting the bend at the summit, nearly 132 gallons per minute if it is wished to include the bend (see Chapter IV., on pipes and siphon pipes); if the head required for the bend were added, if it is of a *large radius* the difference would be practically nothing, but if of small radius it should be added, as it would reduce the discharge.

3. In calculating the quantity of water passed through a sluice *gate* with side walls, as in the Diagram 1, 2, 3, Plate IX., the formula to be used for obtaining the number of gallons per minute with a given head and opening is

$$G = 8 \cdot 025 \times \sqrt{H} \times \cdot 6 \times A \times 6 \cdot 23 \times 60,$$

and for the head required on a certain opening to give a certain discharge,

$$H = \left\{ \frac{\left(\frac{G}{6 \cdot 23 \times 60} \div A \right) \div c}{8 \cdot 025} \right\}^2 .$$

For example.—What will a sluice gate opened 1 foot and 3 feet wide discharge with a head to the *centre* of the orifice of 1 foot? By the formula

Gallons per minute.

$8 \cdot 025 \times \sqrt{1} \times \cdot 6 \times 3 \times 6 \cdot 23 \times 60 = \quad 5399 \cdot 54$

$$\begin{array}{r} 8 \cdot 025 \\ 1 \\ \hline 8 \cdot 025 \\ \cdot 6 \\ \hline 4 \cdot 8150 \\ 3 \\ \hline 14 \cdot 4450 \end{array} \qquad \begin{array}{r} 14 \cdot 4450 \\ 6 \cdot 23 \\ \hline 433350 \\ 288900 \\ 866700 \\ \hline 89 \cdot 992350 \\ 60 \\ \hline 5399 \cdot 541000 \end{array}$$

or a little over 5399½ gallons per minute.

If the head had been required to pass 5399·54 gallons per minute through a sluice opening of 1 foot × 3 feet,

By formula

$$\left\{ \frac{\left(\frac{5399 \cdot 54}{6 \cdot 23 \times 60} \div \text{A} \right) \div c}{8 \cdot 025} \right\}^2 = 1 \text{ foot.}$$

$$\begin{array}{r} 6 \cdot 23 \\ 60 \\ \hline \end{array}$$

$$\begin{array}{l} 373 \cdot 80)5399 \cdot 54(14 \cdot 445 \\ 37380 \\ \overline{166154} \\ 149520 \\ \overline{166340} \\ 149520 \\ \overline{168200} \\ 149520 \\ \overline{186800} \\ 186900 \end{array} \qquad \begin{array}{r} 3)14 \cdot 4450 \\ \hline \cdot 6)4 \cdot 8150 \\ 8 \cdot 025)8 \cdot 025 \\ \hline 1 \cdot 000 \end{array}$$

and the square of 1 is one; therefore the head required is one (1) foot.

The water may occasionally be supplied from a *sluice* valve (see Plate IX., Diagram 4), and then the following formulæ are applicable, assuming that a length of pipe of not more than 2 to 3 diameters is attached to it, for if more pipe is attached, then the friction in the pipe also must be taken into account (see Chapter IV. of this work). Assuming, therefore, that the pipe is not longer than twice or three times the diameter of the *sluice* valve we may use

$$G = \sqrt{H} \times d^2 \times 10$$

$$H = \left(\frac{G}{d^2 \times 10}\right)^2$$

$$d = \sqrt{\left(\frac{G}{\sqrt{H} \times 10}\right)}$$

For example.—What is the discharge of a *sluice* valve 4 inches in diameter with a head on the *centre* of the valve of 1 foot? By the formula*

$$\begin{aligned} &\sqrt{1} \times 4^2 \times 10 \\ &= 1 \times 16 \times 10 \\ &= 16 \times 10 \\ &= 160 \text{ gallons per minute.} \end{aligned}$$

If the head is required to discharge 160 gallons per minute, we have

$$\begin{aligned} &\left(\frac{160}{4^2 \times 10}\right)^2 \\ &= \frac{160}{160} = 1 \text{ foot.} \end{aligned}$$

For *diameter* we have

$$\begin{aligned} &\sqrt{\left(\frac{160}{\sqrt{1} \times 10}\right)} \\ &= \sqrt{\frac{160}{10}} \\ &= \sqrt{16} = 4 \text{ inches.} \end{aligned}$$

Mode of reading the Water Meter.—The water meter index is represented on Plate IX., Diagram 5, and is read as follows:—The pointer (A) has reference to the outer or "tens" circle divided into ten equal parts of 100 gallons each, numbered 100, 200, &c., to 1000, and each divided into ten equal parts of 10 gallons each. The large central hand B has reference to the inner or "thousand" circle divided into ten equal parts of 10,000 gallons each, numbered 10, 20, &c., to 100, and each divided into ten equal parts of 1000 gallons, the whole revolution of the hands showing 100,000 gallons. In addition to these hands, another hand C, complete in a dial of its own, measures 1,000,000 gallons for a complete revolution, each division (of which there are ten) numbered from 1 to 10, showing 100,000 gallons. In order to estimate the quantity of water which has passed through the meter, the various amounts as registered by the various hands must be added together, care being taken only to count completed divisions; thus, in the diagram the hand C has completed one division of 100,000 gallons, the hand B has completed 70 divisions of 1000 gallons each, making 70,000 gallons, the pointer A stands at 500 gallons, and has consequently registered 500 gallons, therefore we have

Hand	C	100,000
„	B	70,000
Pointer	A	500
Total gallons		170,500

170,500 gallons is therefore the quantity which has passed through the meter since all the hands stood at 0. Sometimes another small dial is added registering 10,000,000 gallons, but the principle of adding the registers of each finger and the pointer together remains the same. Where a meter does not stand at 0 when it is fixed, it is necessary to record its reading, this reading is then subtracted from any future reading, and the result is the number of gallons passed since the meter was fixed.

APPENDIX II.

CONTAINING EXAMPLES NOT GIVEN IN THE BODY OF THE WORK.

EXAMPLES FOR SIPHON PIPES.

SUPPOSE a siphon *proper*, with a head of 10 feet, a diameter of 9 inches, and a length of 500 yards, and a bend of 8 feet radius at the top, what is the discharge, the inlet requiring a coefficient of 0·81? First assume a discharge of 200 gallons per minute.

Head for velocity of entry.

Head in feet.

$$\left(\frac{200}{(13\times 9^2)}\right) \qquad = 0\cdot 036$$

For friction.

$$\frac{200^2\times 500}{(3\times 9)^5} \qquad = 1\cdot 393$$

For bend.

$$\left\{0\cdot 131+\left(1\cdot 847\times\left(\frac{4\cdot 5}{96}\right)^{3\cdot 5}\right)\right\}\times\frac{1\cdot 21^2\times 90}{960} = \cdot 001$$

Total head required for 200 gallons per minute .. 1·430

By logarithms, for velocity of entry.

$$\begin{array}{rr}
\text{Log of } 9 = & 0\cdot 9542425 \\
\times\ 2 = & \overline{1\cdot 9084850} \\
\times\ 13 = & 1\cdot 1139434 \\
\text{Log of } (13\times 9^2) & \overline{3\cdot 0224284}
\end{array}$$

$$\begin{array}{rr}
\text{Log of } 200 = & 2\cdot 3010300 \\
\text{Log of } (13\times 9^2) = & 3\cdot 0224284 \\
 & \overline{-\ 1\cdot 2786016} \\
\times\ 2 & 2 \\
 & \overline{-\ 2\cdot 5572032}
\end{array}$$

Number corresponding to log, ·036.

For friction.

$$
\begin{array}{rl}
\text{Log of } 200 = & 2\cdot3010300 \\
\hline
\times\ 2 = & 4\cdot6020600 \\
\times\ 500 = & 2\cdot6989700 \\
\hline
 & 7\cdot3010300 \\
\text{Log of } (3 \times 9)^5 = & 7\cdot1568190 \\
\hline
 & 0\cdot1442110
\end{array}
\qquad
\begin{array}{rr}
\text{Log of } 3 = & 0\cdot4771213 \\
\text{Log of } 9 = & 0\cdot9542425 \\
\hline
 & 1\cdot4313638 \\
\times\ 5 & 5 \\
\hline
 & 7\cdot1568190
\end{array}
$$

Number corresponding to log, 1·393.

For bend.

$$\frac{4\cdot5}{96} = 0\cdot047,$$

and log of this is equal to

$$
\begin{array}{rl}
 & -\ 2\cdot6720979 \\
\hline
\times\ 3\cdot5 = & -\ 5\cdot3523426 \\
\text{Log of } 1\cdot847 = & \ 0\cdot2664669 \\
\hline
 & -\ 5\cdot6188095
\end{array}
$$

Number corresponding to log: 0·00004157

$$
\begin{array}{rl}
\text{Log of } 1\cdot21 = & 0\cdot0827854 \\
\hline
\times\ 2 = & 0\cdot1655708 \\
\times\ 90 = & 1\cdot9542425 \\
\hline
 & 2\cdot1198133 \\
\div\ 960 = & 2\cdot9822712 \\
\hline
 & -\ 1\cdot1375421 \\
\text{Log of } 0\cdot131 = & -\ 1\cdot1172713 \\
\hline
 & -\ 2\cdot2548134
\end{array}
\qquad
\begin{array}{rl}
 & 0\cdot00004157 \\
+ & 0\cdot131 \\
\hline
= & 0\cdot131 \text{ say,}
\end{array}
$$

·01798 number corresponding to log;

then ·01798 ÷ 12 = ·00149.

Then by Rule [59] we have

As $\sqrt{1\cdot43} : \sqrt{10} :: 200 : 528$ gallons per minute.

By logarithms.

$$\text{Log of } 1\cdot43 = 0\cdot1553360$$

$$\div = 0\cdot0776680$$

$$\text{Log of } 10 = 1\cdot0000000$$

$$\div\, 2 = 0\cdot5000000$$

$$\times\, 200 = 2\cdot3010300$$

$$2\cdot8010300$$

$$\div\, \sqrt{1\cdot43} = 0\cdot0776680$$

$$2\cdot7233620$$

528·8 number corresponding to log;

the siphon will therefore discharge 528 gallons per minute.

Suppose, again, we required the diameter of a siphon pipe to discharge 506·38 gallons per minute, that it was 500 yards long, and the bend the same radius, then we must assume a diameter at first; say we assume 6 inches, then we have with this assumed diameter:—

For velocity of entry.

Head in feet.

$$\left(\frac{506\cdot38}{13 \times 6^2}\right)^2 = 1\cdot17$$

For friction.

$$\frac{506\cdot38^2 \times 500}{(3 \times 6)^5} = 67\cdot850$$

For bend.

$$\left\{0\cdot131 + \left(1\cdot847 \times \left(\frac{3}{96}\right)^{3\cdot5}\right)\right\} \times \frac{6\cdot8 \times 90}{960} = 0\cdot001$$

Total head required with 6-inch pipe for 506·38 gallons .. .. = 69·021

By logarithms, for velocity of entry.

$$\text{Log of } 506\cdot38 = 2\cdot7044677$$

$$\text{,,}\ (13 \times 6^2) = 2\cdot6702459$$

$$0\cdot0342218$$

$$\times\, 2 = 0\cdot0684436$$

Number corresponding, 1·17.

For friction.

$$
\begin{aligned}
\text{Log of } 506 \cdot 38 &= 2 \cdot 7044677 \\
\times\, 2 &= 5 \cdot 4089354 \\
\times\, 500 &= 2 \cdot 6989700 \\
& 8 \cdot 1079054 \\
\text{Log of } (3 \times 6)^5 &= 6 \cdot 2763625 \\
& 1 \cdot 8315429
\end{aligned}
$$

Number corresponding, 67·85.

The working of the bend formula is not placed here; an example has before been given. The result is as above, 0·001 feet.

Then by Rule [62] we have

$$\sqrt[5]{10} \;:\; \sqrt[5]{69 \cdot 021} \;::\; 6 \;:\; \overset{\text{Inches.}}{8 \cdot 82} \text{ diameter;}$$

therefore with 10 feet, 500 yards length, and 506·38 gallons per minute discharge, a diameter of say 9 inches is required.

$$
\begin{aligned}
\text{Log. } 10 &= 1 \cdot 0000000 \\
\div\, 5 &= 0 \cdot 2000000 \\
\text{Log. of } 69 \cdot 021 &= 1 \cdot 8389812 \\
\div\, 5 &= 0 \cdot 3677962 \\
\times\, 6 &= 0 \cdot 7781513 \\
& 1 \cdot 1459475 \\
\div\, \sqrt[5]{10} &= 0 \cdot 2 \\
& 0 \cdot 9459475
\end{aligned}
$$

The number of which is 8·82.

It must be remembered that Rule [62] is only strictly applicable when the pipe is a *long* one, or at least 2000 diameters.

Examples for jets.

To what height will a jet rise with 20 feet head on the nozzle, and a nozzle of $\frac{5}{8}$ths of an inch? By Formula [67]

Difference between height of head and height of jet.

$$\frac{20}{5} \times \cdot 0124 \quad = \quad \cdot 0496$$

The jet would therefore rise 19·9504 feet.

What discharge will a $\frac{5}{8}$ths jet give with 20 feet head on the nozzle? By Rule [68]

Gallons per minute.

$$\sqrt{20} \times 5^2 \times 0 \cdot 22$$
$$= 4 \cdot 47 \times 25 \times \cdot 22 \quad = \quad 24 \cdot 585$$

The nozzle would therefore deliver a little over $24\frac{1}{2}$ gallons per minute.

As Formulæ [79] and [80] are of rather a complicated nature, though unavoidably so, to those not used to written formula, I append examples.

In Plate VIII., Figs. 3 and 4, the total width of the river is 80 feet, but by the piers and abutments it is contracted to 60 feet; that is, the spaces A, B, C between the piers only measure 20 feet each, what is the discharge in cube feet per second, the theoretical velocity of approach being equal to 2 feet per second, the depth d_1 equal to 1 foot and the depth d_2 to 2 feet?

1st, the head due to the theoretical velocity of approach of 2 feet per second, corresponding to an actual velocity of about 1·91 foot per second, is

$$\frac{2^2}{64 \cdot 03} \text{ or equal to } \cdot 062 \text{ feet.}$$

Now by Formula [79] we have

$$0 \cdot 6 \times 60 \times 8 \cdot 025 \times \{2 \times \sqrt{(1 + 0 \cdot 062)} + \tfrac{2}{3} \times [(1 + 0 \cdot 062)^{1 \cdot 5} - 0 \cdot 062^{1 \cdot 5}]\}$$
$$= 288 \cdot 9 \times \{2 \cdot 06 + \tfrac{2}{3} \times [1 \cdot 094 - 0 \cdot 015]\}$$
$$= 288 \cdot 9 \times \{2 \cdot 06 + 0 \cdot 719\}$$
$$= 288 \cdot 9 \times 2 \cdot 779$$
$$= 802 \cdot 85 \text{ the discharge in cube feet per second.}$$

Assuming a discharge of 802·85 cubic feet per second, and the depths d_1, d_2 the same, as also h or the head due to the velocity of approach, we have by Formula [80] for the length in lineal feet of the openings added:—

$$\frac{802\cdot85}{\cdot6 \times 8\cdot025 \times \{2 \times \sqrt{(1+0\cdot062)} + \frac{2}{3}[(1+0\cdot062)^{1\cdot5} - 0\cdot062^{\,1\cdot5}]\}}$$

$$= \frac{802\cdot85}{4\cdot815 \times \{2\cdot06 + \frac{2}{3}[1\cdot094 - 0\cdot015]\}}$$

$$= \frac{802\cdot85}{4\cdot815 \times \{2\cdot06 + 0\cdot719\}}$$

$$= \frac{802\cdot85}{4\cdot815 \times 2\cdot779}$$

$$= \frac{802\ 85}{13\cdot38}$$

$$= 60$$

The length of the spaces A, B, C, Diagram 3, is therefore 60 feet, as assumed in Formula [79].

On page 28 I say that the Formula 46 *reversed* should be used when a different coefficient from ·817 is required. For those unaccustomed to these matters it is here given *reversed*, as follows:—

$$\left\{\left(\frac{D}{A} \div c\right) \div 8\cdot025\right\}^2 = H;$$

where

D = discharge in cubic feet per second;
A = area in square feet;
c = coefficient according to circumstances;
H = head in feet.

APPENDIX III.

A FEW remarks are necessary, to make this work complete, on the coefficients to be used when the notches A, shown in Plate I., Diagrams A to I, become short level troughs or level shoots.

First, when the orifice becomes a short trough of about once and a half to twice its width the coefficient rises from ·615 to ·667, and even ·75 under favourable conditions of entry; when the trough, if about 12 inches square, becomes 3 or 4 feet long, the coefficient to be used falls to ·5, and to still less as the length of the trough increases; when, however, the trough is longer than 4 to 5 feet, the formula for *short* channels should be used as shown in Chaper VI.

The coefficients for orifices of the form shown in Plate I., Diagrams A to I, are therefore

For thin plates	·615
For troughs 1½ the width in length	·667 to ·75
For troughs 12 inches square and 3 feet long	·5

The formulæ remaining the same for each form, but the coefficient being altered as above.

Table I.—Discharge over a Weir Twelve Inches Wide.

Coefficient 5·34. Rule:—$\frac{2}{3}\sqrt{\text{Depth in Feet}}$, × 5·34 × Area in Feet = Discharge in Cubic Feet per Second.

Depth in inches.	Cubic feet per second.	Gallons per hour.	Gallons per 24 hours.	Depth in inches.	Cubic feet per second.	Gallons per hour.	Gallons per 24 hours.
1/16	·001338	30	721	2 3/8	·313460	7030	168726
1/8	·003786	85	2038	2 7/16	·325920	7309	175483
3/16	·006955	156	3744	2 1/2	·338490	7591	182199
1/4	·010702	240	5760	2 9/16	·351272	7878	189080
5/16	·014967	336	8056	2 5/8	·364200	8168	196039
3/8	·019675	441	10590	2 11/16	·377292	8462	203086
7/16	·024793	556	13345	2 3/4	·390531	8759	210212
1/2	·030280	679	16292	2 13/16	·403928	9059	217423
9/16	·036133	810	19449	2 7/8	·417475	9363	224715
5/8	·042319	949	22779	2 15/16	·431168	9670	232086
11/16	·048813	1094	26274	3	·444972	9979	239516
3/4	·055638	1248	29948				
13/16	·062734	1407	33768	3 1/16	·458961	10293	247046
7/8	·070100	1572	37732	3 1/8	·473087	10610	254649
15/16	·077745	1744	41848	3 3/16	·487338	10930	262320
1	·085635	1920	46094	3 1/4	·501731	11253	270068
				3 5/16	·516311	11579	277915
1 1/16	·093790	2104	50484	3 3/8	·530997	11909	285821
1 1/8	·102187	2292	55004	3 7/16	·545820	12241	293800
1 3/16	·110825	2485	59653	3 1/2	·560744	12576	301832
1 1/4	·119674	2684	64410	3 9/16	·575833	12914	309955
1 5/16	·128191	2888	69320	3 5/8	·591068	13256	318155
1 3/8	·137617	3086	74075	3 11/16	·606415	13601	326416
1 7/16	·147616	3310	79450	3 3/4	·621906	13948	334755
1 1/2	·157330	3528	84686	3 13/16	·637463	14297	343128
1 9/16	·167270	3751	90037	3 7/8	·653242	14651	351622
1 5/8	·177409	3979	95494	3 15/16	·669108	15006	360161
1 11/16	·187745	4210	101058	4	·685108	15365	368774
1 3/4	·198276	4447	106726				
1 13/16	·208996	4687	112497	4 1/16	·701240	15727	377457
1 7/8	·219879	4931	118355	4 1/8	·717583	16094	386258
1 15/16	·230965	5180	124322	4 3/16	·733855	16459	395013
2	·242213	5432	130376	4 1/4	·750353	16829	403894
				4 5/16	·766975	17201	412841
2 1/16	·253663	5689	136536	4 3/8	·783721	17577	421856
2 1/8	·265277	5949	142791	4 7/16	·800568	17955	430923
2 3/16	·277073	6214	149140	4 1/2	·817554	18336	440077
2 1/4	·289021	6482	155566	4 9/16	·834616	18719	449251
2 5/16	·301164	6754	162108	4 5/8	·851836	19104	458519

Note.—The head to be measured from still water and the weir to have a thin edge and clear overfall. This Table is not applicable where there is velocity of approach.

TABLE I.—DISCHARGE OVER A WEIR TWELVE INCHES WIDE—*continued.*

Depth in inches.	Cubic feet per second.	Gallons per hour.	Gallons per 24 hours.
$4\frac{11}{16}$	·869171	19493	467851
$4\frac{3}{4}$	·886598	19884	477231
$4\frac{13}{16}$	·904178	20278	486694
$4\frac{7}{8}$	·921847	20675	496204
$4\frac{15}{16}$	·939623	21074	505773
5	·957528	21475	515411
$5\frac{1}{16}$	·975547	21879	525110
$5\frac{1}{8}$	·993569	22283	534811
$5\frac{3}{16}$	1·01180	22692	544627
$5\frac{1}{4}$	1·03018	23105	554519
$5\frac{5}{16}$	1·04862	23518	564444
$5\frac{3}{8}$	1·06718	23934	574434
$5\frac{7}{16}$	1·08586	24354	584489
$5\frac{1}{2}$	1·10464	24774	594595
$5\frac{9}{16}$	1·12343	25197	604739
$5\frac{5}{8}$	1·14252	25622	614928
$5\frac{11}{16}$	1·16159	26052	625245
$5\frac{3}{4}$	1·18078	26483	635584
$5\frac{13}{16}$	1·20009	26916	645975
$5\frac{7}{8}$	1·21951	27351	656427
$5\frac{15}{16}$	1·23904	27789	666940
6	1·25865	28229	677498
$6\frac{1}{16}$	1·27838	28671	688116
$6\frac{1}{8}$	1·29821	29116	698791
$6\frac{3}{16}$	1·31813	29563	709511
$6\frac{1}{4}$	1·33814	30012	720288
$6\frac{5}{16}$	1·35828	30463	731122
$6\frac{3}{8}$	1·37851	30917	742012
$6\frac{7}{16}$	1·39884	31373	752960
$6\frac{1}{2}$	1·41926	31831	763946
$6\frac{9}{16}$	1·43967	32289	774936
$6\frac{5}{8}$	1·46042	32754	786100
$6\frac{11}{16}$	1·48113	33219	797251
$6\frac{3}{4}$	1·50195	33686	808456
$6\frac{13}{16}$	1·52279	34153	819674
$6\frac{7}{8}$	1·54380	34624	830984
$6\frac{15}{16}$	1·56489	35097	842340
7	1·58610	35573	853754
$7\frac{1}{16}$	1·60740	36051	865219
$7\frac{1}{8}$	1·62879	36530	876735
$7\frac{3}{16}$	1·65029	37012	888302
$7\frac{1}{4}$	1·67187	37496	899919
$7\frac{5}{16}$	1·69354	37983	911585
$7\frac{3}{8}$	1·71530	38471	923301
$7\frac{7}{16}$	1·73716	38961	935065
$7\frac{1}{2}$	1·75911	39453	946879
$7\frac{9}{16}$	1·78114	39947	958738
$7\frac{5}{8}$	1·80328	40444	970654
$7\frac{11}{16}$	1·82550	40942	982617
$7\frac{3}{4}$	1·84781	41444	994672
$7\frac{13}{16}$	1·87021	41945	1006680
$7\frac{7}{8}$	1·89272	42449	1018790
$7\frac{15}{16}$	1·91530	42956	1030950
8	1·93797	43464	1043150
$8\frac{1}{16}$	1·96072	43975	1055400
$8\frac{1}{8}$	1·98359	44488	1067710
$8\frac{3}{16}$	2·00650	45000	1080018
$8\frac{1}{4}$	2·02954	45518	1092442
$8\frac{5}{16}$	2·05264	46036	1104880
$8\frac{3}{8}$	2·07585	46557	1117368
$8\frac{7}{16}$	2·09914	47079	1129907
$8\frac{1}{2}$	2·12252	47604	1142494
$8\frac{9}{16}$	2·14598	48130	1155122
$8\frac{5}{8}$	2·16952	48683	1167791
$8\frac{11}{16}$	2·19313	49187	1180501
$8\frac{3}{4}$	2·21686	49719	1193273
$8\frac{13}{16}$	2·24066	50253	1206086
$8\frac{7}{8}$	2·26453	50789	1218934
$8\frac{15}{16}$	2·28823	51320	1231690
9	2·31229	51860	1244640
$9\frac{1}{16}$	2·33644	52401	1257640
$9\frac{1}{8}$	2·36064	52944	1270665
$9\frac{3}{16}$	2·38485	53487	1283700
$9\frac{1}{4}$	2·40930	54036	1296856
$9\frac{5}{16}$	2·43374	54571	1309714
$9\frac{3}{8}$	2·45826	55134	1323212
$9\frac{7}{16}$	2·48293	55687	1336490
$9\frac{1}{2}$	2·50759	56240	1349764
$9\frac{9}{16}$	2·53239	56796	1363113
$9\frac{5}{8}$	2·55751	57359	1376636
$9\frac{11}{16}$	2·58223	57914	1389932
$9\frac{3}{4}$	2·60727	58475	1403419
$9\frac{13}{16}$	2·63237	59068	1416930

TABLE I.—DISCHARGE OVER A WEIR TWELVE INCHES WIDE—*continued.*

Depth in inches.	Cubic feet per second.	Gallons per hour.	Gallons per 24 hours.
9 7/8	2·65758	59604	1430500
9 15/16	2·68285	60170	1444103
10	2·70815	60738	1457722
10 1/16	2·73364	61310	1471440
10 1/8	2·75906	61880	1485124
10 3/16	2·78467	62454	1498906
10 1/4	2 81033	63030	1512720
10 5/16	2·83610	63607	1526591
10 3/8	2·86192	64187	1540493
10 7/16	2·88783	64768	1554437
10 1/2	2·91380	65350	1568416
10 9/16	2·93983	65934	1582427
10 5/8	2·96596	66520	1596492
10 11/16	2·99218	67108	1610607
10 3/4	3·01847	67698	1624756
10 13/16	3·04485	68289	1638950
10 7/8	3·07126	68882	1653170
10 15/16	3·09778	69476	1667445
11	3·12439	70073	1681770
11 1/16	3·15071	70664	1695939
11 1/8	3·17774	71270	1710487
11 3/16	3·20456	71873	1724960
11 1/4	3·23151	72476	1739430
11 5/16	3·25845	73080	1753931
11 3/8	3·28552	73687	1768502
11 7/16	3·31263	74295	1783100
11 1/2	3·33980	74905	1797722
11 9/16	3·36703	75515	1812380
11 5/8	3·39443	76130	1827125
11 11/16	3·42180	76744	1841860
11 3/4	3·44931	77360	1856661
11 13/16	3·47686	77978	1871490
11 7/8	3·50451	78599	1886380
11 15/16	3·53220	79220	1901287
12	3·55996	79842	1916230
12 1/4	3·67195	82354	1976500
12 1/2	3·78481	84886	2037260
12 3/4	3·89889	87444	2098661
13	4·01431	90033	2160790
13 1/4	4·13042	92637	2223290
13 1/2	4·24790	95272	2286535
13 3/4	4·36667	97933	2350400
14	4·48608	100613	2414720
14 1/4	4·60679	103321	2479706
14 1/2	4·72876	106056	2545360
14 3/4	4·85137	108805	2611338
15	4·97528	111585	2678052
15 1/4	5·10024	114388	2745316
15 1/2	5·22593	117207	2812972
15 3/4	5·35300	120057	2881370
16	5·48108	122929	2950313
16 1/4	5·60983	125809	3019614
16 1/2	5·73978	128731	3089562
16 3/4	5·87099	131674	3160190
17	6·00264	134627	3231053
17 1/4	6·13563	137609	3302635
17 1/2	6·26967	140615	3374780
17 3/4	6·40427	143640	3447240
18	6·54001	146679	3520301
18 1/4	6·67692	149750	3594000
18 1/2	6·81408	152826	3667825
18 3/4	6·95318	155945	3742701
19	7·09285	159078	3817880
19 1/4	7·23297	162221	3893304
19 1/2	7·37449	165395	3969480
19 3/4	7·51696	168590	4046170
20	7·65985	171795	4123080
20 1/4	7·80392	175026	4200633
20 1/2	7·94911	178282	4278782
20 3/4	8·09464	181546	4357120
21	8·24154	184841	4436190
21 1/4	8·38933	188155	4515740
21 1/2	8·53762	191481	4595562
21 3/4	8·68687	194829	4675901
22	8·83736	198204	4756904
22 1/4	8·98812	201585	4838051
22 1/2	9·14001	204992	4919810
22 3/4	9·29313	208426	5002230
23	9·44643	211865	5084770
23 1/4	9·60091	215329	5167900
23 1/2	9·75635	218815	5251570
23 3/4	9·91218	222310	5335450
24	10·0694	225834	5420031

TABLE II.—SHOWING COEFFICIENTS OF DISCHARGE FROM SQUARE AND DIFFERENTLY PROPORTIONED RECTANGULAR LATERAL ORIFICES IN THIN VERTICAL PLATES FROM PONCELOT AND LESBROS. THE ORIFICES ARE ALL 7·874 INCHES WIDE.

Heads of Water measured to the upper sides of the orifices, in inches.	Ratio of the head to the length of the orifice.	Square orifice 7·874″ × 7·874″. Ratio of sides 1 to 1. Head taken back from the orifice.	Rectangular orifice 7·874″ × 3·937″. Ratio of sides 2 to 1. Head taken back from the orifice.	Rectangular orifice 7·874″ × 1·968″. Ratio 4 to 1. Head taken back from the orifice.	Rectangular orifice 7·874″ × 1·181″. Ratio 7 to 1. Head taken back from the orifice.	Rectangular orifice 7·874″ × 0·787″. Ratio 10 to 1. Head taken back from the orifice.	Rectangular orifice 7·874″ × 0·394″. Ratio 20 to 1. Head taken back from the orifice.	REMARKS, &c.
0	..	..	..	..	..	..	..	
0·197	·025	..	..	..	..	..	·705	
0·394	·050	..	..	·607	·630	·660	·701	
0·591	·075	..	·593	·612	·632	·660	·697	
0·787	·100	·572	·596	·615	·634	·659	·694	
1·181	·150	·578	·600	·620	·638	·659	·688	
1·575	·200	·582	·603	·623	·640	·658	·683	
1·969	·250	·585	·605	·625	·640	·658	·679	
2·362	·300	·587	·607	·627	·640	·657	·676	
2·756	·350	·588	·609	·628	·639	·656	·673	
3·150	·400	·589	·610	·629	·638	·656	·670	
3·545	·450	·591	·610	·629	·637	·655	·668	

3·937	·500	·592	·611	·630	·637	·654	·666
4·724	·600	·593	·612	·630	·636	·653	·663
5·512	·700	·595	·613	·630	·635	·651	·660
6·299	·800	·596	·614	·631	·634	·650	·658
7·087	·900	·597	·615	·630	·634	·649	·657
7·874	1·000	·598	·615	·630	·633	·648	·655
9·843	1·250	·599	·616	·630	·632	·646	·653
11·811	1·500	·600	·616	·629	·632	·644	·650
15·748	2·000	·602	·617	·628	·631	·642	·647
19·685	2·500	·603	·617	·628	·630	·640	·644
23·622	3·000	·604	·617	·627	·630	·638	·642
27·560	3·500	·604	·616	·627	·629	·637	·640
31·497	4·000	·605	·616	·627	·629	·636	·637
35·434	4·500	·605	·615	·526?	·628	·634	·635
39·371	5·000	·605	·615	·626	·628	·633	·632
43·307	5·500	·604	·614	·625	·627	·631	·629
47·245	6·000	·604	·614	·624	·626	·628	·626
51·182	6·500	·603	·613	·622	·624	·625	·622
55·119	7·000	·603	·612	·621	·622	·622	·618
59·056	7·500	·602	·611	·620	·620	·619	·615
62·993	8·000	·602	·611	·618	·618	·617	·613
66·930	8·500	·602	·610	·617	·616	·615	·612
70·867	9·000	·601	·609	·615	·615	·614	·612
74·805	9·500	·601	·608	·614	·613	·612	·611
78·742	10·000	·601	·607	·613	·612	·612	·611
118·112	15·000	·601	·603	·606	·608	·610	·609

TABLE III.

The following table exhibits the results of experiments made on the canal at Languedoc by Lespinasse, on a sluice gate of a breadth of 4·265 feet. The woodwork which surrounded this orifice was 0·886 foot thick, and even 1·772 foot thick on the lower edge, also when the gate was raised only slightly the contraction ceased on all four sides, and the coefficient increased considerably; for example, the gate being only raised 0·394 foot, had for a coefficient 0·803,* while with 1·509 foot opening a coefficient of only 0·641 was obtained.

Openings.		Head on the centre of orifice.	Discharge in one second, cube feet.	Coefficient.
Area, sq. feet.	Height, feet.			
7·745	1·805	14·554	145·292	·613
6·992	1·640	6·631	92·635	·641
6·992	1·640	6·247	88·221	·629
6·466	1·509	12·878	138·937	·641
6·723	1·575	13·586	128·764	·647
6·723	1·575	6·394	83·948	·616
6·723	1·575	6·217	79·857	·594
6·717	1·575	6·480	85·219	·621
			Mean coefficient ..	·625

* This was probably due to the orifice becoming a rectangular tube when slightly raised.

TABLE IV.—SHOWING THE COEFFICIENTS OF DISCHARGE FOR *Short Tubes* WITH SQUARE EDGES NEXT RESERVOIR.

Name of Observer.	Tube.		Head, feet.	Coefficient.	REMARKS.
	Diameter, feet.	Length, feet.			
Castel	·0509	·1312	·6562	·827	
"	·0509	·1312	1·5749	·829	
"	·0509	·1312	3·2478	·829	
"	·0509	·1312	6·5620	·829	
"	·0509	·1312	9·9414	·830	
Eytelwein ..	·0853	·2559	2·3623	·821	
Bossut	·0886	·0341	12·6318	·804	
"	·0886	·1772	12·6975	·804	
"	·0886	·3543	12·8615	·804	
Venturi.. ..	·1345	·4200	2·8873	·822	
Michelotti ..	·2658	·7087	7·1526	·815	
"	Square ·2658	·7087	12·4678	·803	
"	·2658	·7087	22·0155	·803	
			Mean coefficient ..	·817	

TABLE V.—COEFFICIENTS OF DISCHARGE FOR *Circular* ORIFICES IN THIN PLATES.

Name of Observer.	Diameter in inches.	Head in feet.	Coefficient.	Ratio of head to diameter.	REMARKS.
Mariotti ..	0·268	5·873	0·692	263·000	
" ..	0·268	25·920	0·692	1160·600	
Castel	0·394	2·133	0·673	64·964	
"	0·394	1·017	0·654	30·974	
"	0 590	0·453	0·632	9·213	
"	0·590	0·984	0·617	20·015	
Eytelwein ..	1·027	2·372	0·618	27·715	
Bossut	1·067	4·265	0·619	47·965	
Michelotti ..	1·067	7·317	0·618	92·290	
Castel	1·181	0·223	0·629	1·477	
Venturi.. ..	1·614	2·887	0·622	21·464	
Bossut	2·126	12·500	0·618	70·555	
Michelotti ..	2·126	7·218	0·607	40·741	
" ..	3·189	7·349	0·613	27·650	
" ..	3·189	12·500	0·612	47·	
" ..	3·189	22·179	0·597?	83·	
" ..	6·378	6·923	0·619	13·	
" ..	6·378	12·008	0·619	22·	
Mean coefficient excluding 0·597		..	0·632		

TABLE VI.—COEFFICIENTS OF DISCHARGE FOR *Square* ORIFICES IN THIN PLATES.

Name of Observer.	Side of square, in inches.	Head, in feet.	Coefficient.	Ratio of head to length.	REMARKS.
Castel	0·394	0·164	0·655	5	
Bossut	1·063	12·500	0·616	141	
Michelotti ..	1·063	12·500	0·607	141	
„ ..	1·063	22·409	0·606	253	
Bossut	2·126	12·500	0·618	70	
Michelotti ..	2·126	7·349	0·603	41	
„ ..	2·126	12·566	0·603	70	
„ ..	2·126	22·245	0·602	125	
„ ..	3·228	7·415	0·616	28	
„ ..	3·189	12·566	0·619	47	
„ ..	3·189	22·376	0·616	84	
	Mean coefficient ..		0·615	..	

TABLE VII.—SHOWING THE DISCHARGES THROUGH A SHORT PIPE OF TWO DIAMETERS COMING FROM A BOX, AS SHOWN ON DIAGRAM 1, PLATE VIII., WITH A HEAD ON THE *Centre* OF THE PIPE * OF ONE (1) FOOT.

Diameter in inches.	Discharge in gallons per minute.	Discharge in gallons in 8 hours.	REMARKS.
½	3·25	1,560	The head assumed to give the discharge shown is in all cases *one foot* in this table. The discharge for any other head may be found by the rule that the discharge is proportional to the square root of the head. A sluice head in Victoria is equal to about 200,000 gallons in 24 hours, therefore a 4-inch pipe in a box of the description shown in Plate VIII. should have a head of 0·668 feet or $8\frac{1}{32}$ inches over the *centre* of the pipe to give one "sluice head." The head should be measured to still water—or an addition made for velocity of approach.
1	13·00	6,240	
1½	29·25	14,040	
2	52·00	24,960	
2½	81·25	39,000	
3	117·00	56,160	
3½	159·25	76,440	
4	208·00	99,840	
4½	263·25	126,360	
5	325·00	156,000	
5½	393·25	188,760	
6	468·00	224,640	
6½	549·25	263,640	
7	637·00	305,760	
7½	731·25	351,000	
8	832·00	399,360	
8½	939·25	450,840	
9	1053·00	505,440	
9½	1173·25	563,160	
10	1300·00	624,000	
10½	1433·25	687,960	
11	1573·00	755,040	
11½	1719·25	825,240	
12	1872·00	898,560	

* This pipe is supposed to be horizontal.

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